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STRUCTURAL STABILITY EVALUATION GULL LAKE DAM

by

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June 1981

Final Report

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20. ABSTRACT (*Continue on reverse side if necessary and identify by block number*)

A typical interior monolith at Gull Lake Dam was evaluated for adequacy in stability for five load cases:

(1) Normal operation
(2) Normal operation with truck loading (H15-44)
(3) Normal operation with earthquake
(4) Normal operation with ice
(5) High-water condition.

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20. ABSTRACT (Continued)

By using the conventional stability analysis (rigid body assumptions), the approximate magnitude of loads on the foundation piles were determined; but, without knowing the supporting characteristics of the foundation material, the adequacy of the structure's stability was inconclusive.

The supporting characteristics of the foundation material were determined by in situ testing using a pressuremeter to predict the horizontal supporting characteristics of the pile-soil system.

Two 4-in.-diam core holes were drilled through typical monoliths to obtain access to the foundation material. The pressuremeter tests were performed and in situ soils data obtained. The in situ data were used to obtain the supporting characteristics of the foundation material. The horizontal modulus of subgrade reaction was obtained for three test positions in both of the two test holes. It was obtained as a variation of horizontal pressure and depth within the foundation material.

The horizontal modulus of the subgrade reaction was then used in a three-dimensional direct stiffness analysis of the piling foundation. A beam on elastic foundation analysis was performed and the pressure, moment, and deflection along the length of the most critically loaded pile was determined.

It was determined that the horizontal and vertical forces on the piling were not excessive in relation to the material properties of the pile. The pile deflections were also adequate under all load conditions.

Unconfined compressive tests on the concrete gave an average strength of 5550 psi, which is adequate for this type of structure. Core logs are presented that give evidence that the concrete is well consolidated. A petrographic analysis of the concrete showed that it was nonair-entrained; the voids were commonly filled with ettringite; and small amounts of alkali-silica reaction product were present. These characteristics are not considered detrimental for this structure and exposure conditions.

The structure is adequate in stability; the concrete is of good quality, and with rehabilitation of the deteriorated surface, the concrete dam can be expected to perform satisfactorily for many more years.

PREFACE

The evaluation of the stability of Gull Lake Dam was conducted for the U. S. Army Engineer District, St. Paul, by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES). Authorization for this investigation was given in Intra-Army Order for Reimbursable Services No. NCS-1A-78-75, dated 23 July 1979.

The contract was monitored by the St. Paul District with assistance from Messrs. Roger Ronning and Jerry Blomker. Their cooperation and assistance were greatly appreciated.

The study was performed under the direction of Messrs. Bryant Mather, W. J. Flathau, and J. M. Scanlon, Jr., SL. The structural stability analysis was performed by Dr. Carl Pace and Mr. Roy Campbell. The core logging and writing of the petrographic report was performed by Miss Barbara Pavlov and Mr. Sam Wong under the technical supervision of Mr. Alan Buck. The testing was performed by Mr. Mike Lloyd. The computer programming by Miss Alberta Wade was appreciated. The core drilling was under the direction of Mr. Mark Vispi, Geotechnical Laboratory, WES. Dr. Pace prepared the report.

Commanders and Directors of WES during the conduct of the program were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

U. S. inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1233.489	cubic metres
cubic inches	0.00001638706	cubic metres
feet	0.3048	metres
inches	0.0254	metres
inches per pound	0.00571015	metres per newton
kips (force)	4448.222	newtons
kip·feet	1355.818	newton·metres
kips (force) per square foot	47.88026	kilopascals
miles (U. S. statute)	1609.344	metres
pounds (force)	4.448222	newtons
pounds (mass)	0.4535924	kilograms
pounds (force) per square inch	6.894757	kilopascals
pounds per inch	175.1268	newtons per metre
square miles	2.589988	square kilometres

STRUCTURAL STABILITY EVALUATION

GULL LAKE DAM

PART I: INTRODUCTION

Background

1. Gull Lake Dam is located on the Gull River, 1006.4 miles* above the mouth of the Ohio River, about one-half mile below the outlet of Gull Lake, and about 11 miles northwest of Brainerd, Minn., in the extreme southern portion of Cass County (Figure 1). By river it is approximately 165 miles above St. Paul, Minn.

2. Gull Lake Dam is a low-head dam consisting of a gated concrete control structure and short earth-filled tieback embankment section. The dam was built in 1911-1913 and is the newest in the Mississippi River headwaters reservoir system. The dam was built to produce a reservoir with the primary purpose of storing water to improve navigation on the Mississippi River between St. Paul and Lake Pepin. During past floods, the upper operating limit of the structure has been exceeded eight times, but the flowage limit has never been exceeded.

3. The total drainage area above the dam is 287 square miles. The reservoir at the maximum operating stage of 7.0 ft has an area of about 20 square miles. Gull River is about 35 miles in length from its headwaters in Sibley Lake (T. 136 N., R 29 W.) to its junction with the Crow Wing River about 11 miles below Gull Lake Dam. Numerous lakes and streams are tributaries to the Gull River. The project map of the Mississippi Reservoirs is presented in Figure 2.

4. Demands by resort and private property owners in the Gull Reservoir area resulted in revised regulations that have reduced the usable storage capacity of the reservoir by limiting its drawdown. During periods of abnormally high inflow, storage is utilized up to the

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

7.0-ft stage. Flowage rights have been acquired to an 11.0-ft stage to allow for wave action and seepage damage. Stored water is released, if required, during the summer to augment streamflows for water supply, water power, or other beneficial uses. To provide storage capacity for the spring runoff, the reservoir is lowered during the winter months to reach a stage of 5.0 ft by April 1. Outflow during the spring ice breakup period depends on the amount of runoff available for filling the reservoir and downstream conditions. General reservoir data are presented in Table 1. More detailed information can be obtained from U. S. Army Engineer District, St. Paul (1973, 1977).

Project Features

5. The dam consists of a gated concrete control structure and short earth-filled tieback embankment dikes. The right bank dike is 129 ft in length with 90 ft of curtain wall. The left bank is 72 ft in length with 33 ft of curtain wall. Top elevation of both dikes is 1197.75.* The top of the dam has a public roadway with top elevation of 1198.97 ft.

6. The control structure is of reinforced concrete, supported on timber piling. There are five sluiceways, each 5 ft wide. In addition, there are an 11-ft log sluice and a 5-ft fishway in the structure. The total length of the project between the abutments is 68 ft 11 in.

7. Operation of the dam was by use of stop logs prior to 1971 when steel slide gates were installed. The bridge over the control structure was also reconstructed and resurfaced in 1971. Other than these changes, the structure is basically as originally constructed. A drawing showing the general construction of the dam is presented in Figure 3. Views of the dam piers are presented in Figure 4. General data concerning the dam are presented in Table 2.

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* All elevations are referred to msl, 1929 adjustment.

Objective

8. The objective of this study is to evaluate the stability of the concrete control structure. For this evaluation two cores were drilled through the dam and into the foundation. The foundation material was tested in situ for a determination of its supporting capabilities. The cores were examined, and the structural stability of the dam was evaluated. The stability analysis was performed in accordance with current Corps of Engineer criteria.

Scope

9. This study is limited to a structural stability evaluation of the concrete control structure with consideration given to foundation and concrete properties.

PART II: CORING PROGRAM

10. Since Gull Lake Dam is a low-head structure and is only 68 ft 11 in. from abutment to abutment, limited coring was performed to obtain the properties of the concrete and to obtain access to the foundation material. The two 4-in. cores were drilled through piers 2 and 4 from the roadway. Looking from upstream to downstream, each core hole was 13.71 ft from the upstream end of the pier and 2 ft from the left side of the pier toward its center.

11. A truck-mounted drill rig, using diamond core bits and 5-ft-long double-tube swivel-head core barrels, was used to obtain the cores from the concrete. A slotted casing was used to house a pressuremeter probe as the casing was driven to the desired depth for pressuremeter tests.

12. The coring program was oriented toward determining:

- a. Depth of deteriorated concrete.
- b. Uniformity of concrete with depth.
- c. Unconfined compressive strength of the concrete, and
- d. To make a hole through the structure to give access to the foundation in order that in situ tests and properties could be obtained for the foundation.

Undisturbed samples of foundation material were obtained and standard penetration tests were performed on this material.

13. The in situ strength of the foundation material was an important factor in the stability analysis of the dam, which is supported on timber piles embedded in the foundation material.

14. The coring program was considered a minimum for obtaining representative information on the concrete and foundation material but was adequate for this particular dam. The core holes were not grouted, and a capped pipe was used to seal the top opening in order that in the future the holes could be used for obtaining piezometric data. Pictures of the cores from holes G-P2 and G-P4 are presented in Figure 5, and a closer view of a core and a cut section from hole G-P4 is presented in Figure 6. The concrete at Gull Lake Dam was found to be very uniform.

PART III: PETROGRAPHIC REPORT AND CORE LOGS

Samples

15. Two 4-in.-diam concrete cores were received on 29 October 1979 for tests and examination. The cores were taken from two 67-year-old concrete piers at Gull Lake Dam. A description of the cores is given below:

Core No.	Location	Elevation	Length
G-P2	Pier 2	1198.95 ft	13.0 ft
G-P4	Pier 4	1198.95 ft	13.0 ft

Test Procedures

16. The two cores were logged in the laboratory, and samples were selected for physical testing and petrographic examinations. The specimens for physical testing were taken from the upper, middle, and lower portions of each core. This was also generally true for the petrographic specimens.

17. One of the petrographic samples from core G-P4 was selected to be typical of the concrete in both cores. This piece was sawed longitudinally, and one of the sawed surfaces was then ground smooth. This smooth surface was examined with a stereomicroscope. Freshly broken surfaces of concrete from both cores were also examined with a stereomicroscope.

18. A cement paste concentrate was made from core G-P4. The cement paste was extracted from a typical piece of concrete by breaking up the concrete with a mortar and pestle. The broken material was passed over a No. 100 sieve. The material passing the sieve was then ground to pass a No. 325 sieve. This powder was then examined by X-ray diffraction.

19. A white reaction product found coating some aggregate surfaces was examined with a stereomicroscope and as an oil immersion mount

with a polarizing microscope. An X-ray diffraction pattern was also made with an X-ray diffractometer using nickel-filtered copper radiation.

Results

20. The logs of the two cores are shown in Figures 7 and 8. The concrete in both cores was similar; it was non air-entrained, showed good consolidation, and the presence of underside voids on the aggregate particles suggested the water content had been somewhat high. Maximum aggregate size was judged to be about 2 in. The coarse aggregate appeared to be crushed particles that were granite, or granite gneiss, or dark colored, fine-grained igneous rocks. The tops of both cores were overlaid with a wood deck that was covered with a sand and gravel mixture and then topped with a layer of asphalt. The extent of the nonconcrete cover is shown in the logs (Figures 7 and 8).

21. The test specimens were selected to represent typical concrete. No concrete was considered to be of significantly lower quality by appearnace.

22. Ettringite was commonly found partially filling voids in the concrete. Small amounts of white alkali-silica reaction gel were found on a few aggregate surfaces. While this occurrence was noted more often on the dark particles, it was also found on some of the light-colored particles. A powder immersion mount of this gel showed it to have a refractive index below 1.498. The presence of this gel indicated that some alkali-silica reaction had occurred; the lack of cracking indicated that this reaction had not damaged the concrete that was examined.

23. The X-ray diffraction pattern of the cement paste concentrate showed calcium hydroxide, ettringite, tetracalcium aluminate carbonate-11-hydrate (monocarboaluminate), and calcite as paste compounds along with quartz, plagioclase, and potassium feldspars from the aggregate. While the calcite was assumed to be present due to carbonation of the calcium hydroxide, it may be an aggregate constituent, or it may be present in both the aggregate and the paste.

Discussion

24. The concrete from both cores was intact except for horizontal cold joints formed in both cores at depths of 5.3 and 11.3 ft. Although the concrete was not air-entrained and did show evidence that some alkali-silica reaction had occurred, there was no significant cracking or deterioration of the concrete. Therefore, it was concluded that the concrete from this structure, as judged by these cores, was in good physical condition.

PART IV: FOUNDATION AND CONCRETE PROPERTIES

In Situ Foundation Testing

25. Undisturbed samples of the soil under Gull Lake Dam could not be readily obtained. Hence, in situ testing of foundation material supporting the piling was accomplished to obtain the structural supporting characteristics of the pile-foundation system.

26. The pressuremeter method was used to measure deformation properties and obtain a rupture or limit resistance of the foundation material.

Pressuremeter Field Tests and Results

27. To test the foundation under the dam piers at Gull Lake, access to the foundation material had to be obtained. This was done by coring a 4-in. hole through the dam piers and down to the foundation material. Then, a slotted casing containing the pressuremeter probe was driven to the desired depth in the foundation material. A pressurized bottle of gas was used as the pressure source. The pressuremeter test was performed at three elevations in each test hole. The location of the probe below the bottom of the pier is presented in Table 3 for each test hole.

28. The locations of the probe during testing were at depths in the foundation material that would give representative data from which the supporting capacity of pile and foundation material could be determined.

29. Standard penetration (split spoon) tests were performed above and below the pressuremeter test locations. The split spoon data are presented in Table 4.

30. Disturbed samples of the foundation material were obtained and transported to the laboratory for classification. The foundation material under Gull Lake Dam is mainly a silty sand (Figures 9 through 12).

31. The main characteristic of the material for evaluating horizontal support to the wood piles is the subgrade modulus, and its variation with pressure and depth in the foundation. The pressuremeter tests determined these data. Plots of data for hole G-P2 are presented in Figures 13 through 19 and for G-P4 in Figures 20 through 26.

32. The actual pressure which is applied to the probe is higher than that which is read at the control unit due to the hydrostatic pressure of the water in the tubing. On the other hand, the pressure applied to the soil is less than the probe pressure due to the resistance of the rubber membrane. The corrected pressure curves are presented in Figures 13, 14, and 15 for hole G-P2 and in Figures 20, 21, and 22 for hole G-P4.

33. Several methods were used in obtaining the limit pressure of the foundation material, but it was found that extrapolating the curves out to two times the initial volume of the cavity gave excellent results. The limit pressures were obtained and are presented in Figures 15 and 22 for hole G-P2 and G-P4, respectively.

34. The shear modulus (G) depends not only on the slope of the pressure-volume curve but also on the volume of the probe. The average volume used in calculating the shear modulus is as follows:

$$G_M = \left[535 + \frac{V(I) + V(I + 1)}{2}\right] \frac{\Delta P}{\Delta v} \tag{1}$$

$$= \left[535 + \frac{V(I) + V(I + 1)}{2}\right]\left[\frac{P(I + 1) - P(I)}{V(I + 1) - V(I)}\right]$$

The deformation modulus, which is something roughly equivalent to Young's modulus, is obtained from the well-known relation:

$$G_M = \frac{E_p}{2(1 + v)} \tag{2}$$

Poisson's ratio is used as 0.33, and the resulting deformation modulus is called the Ménard modulus, E_m.

$$E_m = 2(1 + v)G_M \tag{3}$$

$$= 2(1 + 0.33)G_M = 2.66G_M$$

The Ménard modulus is presented in Figures 17 and 24 for hole G-P2 and G-P4, respectively.

35. The horizontal subgrade modulus (k) is obtained from the following equations:

$$\frac{1}{k} = \frac{2}{9E_M} \cdot B_o\left(\frac{B}{B_o} \times 2.65\right)^{\alpha} + \frac{\alpha}{6E_M} \cdot B \qquad (B > 0.6 \text{ m}) \tag{4}$$

$$\text{or} \quad \frac{1}{k} = \frac{B}{E_M}\left(\frac{4(2.65)^{\alpha} + 3\alpha}{18}\right) \qquad (B < 0.6 \text{ m}) \tag{5}$$

where

B_o = reference pile diameter, 0.6 m

B = pile diameter

α = rheological coefficient given in Figures 3-48 of Baguelin, Jiziquel, and Shields (1978).

36. After a representative value of k has been determined, it can be multiplied by the pile diameter to obtain the horizontal modulus of reaction for the pile-soil system. The horizontal modulus of reaction of the soil can be used in the piling analysis to obtain deflections, forces, and moments to use in evaluating the adequacy of the pile foundation.

Piling and Concrete Data

37. The 12-in.-diam Norway Pine pilings, which support the monoliths at Gull Lake, are approximately 15 ft long. The properties of the 12-in. Norway Pine piling are as follows:

Modulus of elasticity (E) = 1.32×10^6 psi

Shear modulus (G) = 0.45×10^6 psi

Allowable compressive stress parallel to grain = 1100 psi

Allowable tensile stress parallel to grain = 775 psi

Allowable average shear stress = 75 psi

Allowable compressive load on a pile = 124 kips

Allowable tensile load on a pile = 0 kips

Average allowable lateral load per pile = 8.5 kips

Allowable moment in a pile = 131,000 in.-lb or 10.9 kip-feet.

38. The properties of Norway wood can be found in many handbooks. One such reference is presented (Southern Pine Association, 1954).

39. The unconfined compressive strength of the concrete is presented in Table 5. The average unconfined compressive strength is 5500 psi, which is excellent. Since the interior concrete is of good quality, the deteriorated surface concrete should be repaired before the deterioration increases in depth. The surface concrete needs to be rehabilitated to eliminate water from entering cracks and freezing causing accelerated deterioration.

40. There are a number of methods of repair that might be used; but, the upper headwater structures are ideal for an economical repair such as:

a. clean surface concrete.

b. fill cracks.

c. paint on a cementitious coating to rehabilitate the surface concrete.

This type repair can be performed rapidly and economically. It is analogous to cleaning and filling cracks and then painting a room in a house. Any local labor could do the work with only common tools.

41. Under some conditions, an acrylic-polymer coating mix of a composition as listed in Table 6 and Table 7 might be used. Certain polymers have exhibited good bond and noncracking when used in ordinary environments. They have also shown good resistance to freezing-and-thawing environments. The particular polymer to be used should be tested as follows before being used to rehabilitate the surface concrete of the Upper Mississippi Headwater Structure.

a. Determine the resistance of the coating to cracking during extreme temperature changes.

b. Determine its ability to retain bond capability in freezing-and-thawing environments.

c. Determine its ability to "breathe" (allow water to escape from the interior concrete through the coating).

PART V: STABILITY ANALYSIS

Conventional Stability Analysis

42. Conventional stability analysis is very effective in obtaining the distribution of forces to the piles within a pile group, considering the structure and pile system rigid. This does not account for load distributions due to pile and structure deformations or for the strength and supporting characteristics of the soil on the piling system. To use the conventional analysis, field tests must have been performed evaluating and determining allowable loads on the particular piling that would be supporting the dam monoliths. The flexibility of the base of the structure can be taken into account in obtaining the distribution of loads to the piling, but for Gull Lake Dam the monoliths are of such size and shape that a rigid base analysis is adequate.

43. The geometry and loadings on a particular interior monolith of Gull Lake Dam are presented in Figures 27 through 31. Five load cases were analyzed:

- a. Normal operation.
- b. Normal operation with truck loading (H15-44).
- c. Normal operation with earthquake.
- d. Normal operation with ice.
- e. High-water condition.

The loading and analysis determining the applied forces and moments on the pile layout are also presented in Figures 27 through 31. The results of the conventional piling analysis are presented in Table 8.

44. At this point, allowable vertical and horizontal loads had to be determined to evaluate the adequacy and stability of the piling foundation. These allowables were not known for Gull Lake Dam. The following approach was considered the most efficient and economical to evaluate the adequacy of the piling foundation at Gull Lake Dam.

- a. Core through the monoliths and obtain samples of concrete for evaluation and in the process gain access to the foundation material.

b. Perform in situ pressuremeter tests as described in Part IV.

c. Use the pressuremeter test results to obtain a modulus of subgrade reaction for the foundation material to use in the stability analysis of the pile foundation.

d. Obtain allowable vertical and horizontal loads, considering the material properties of the pile. They would be used to evaluate the pile under actual applied loads.

e. The adequacy of the pile foundation, considering the strength characteristics of the foundation material, would be based on deflections at the top of the pile. If the deflection of the pile in a horizontal direction is less than one quarter inch, the piling system would be considered adequate.

45. At Gull Lake Dam a conservative constant subgrade modulus would be used. The variation in subgrade modulus with depth into the foundation (Figures 19 and 26) does not show a distinct pattern that justifies using a variable subgrade modulus. Hole G-P2 indicates an increase in subgrade modulus with depth (Figure 19), but the data from hole G-P4 do not indicate this variation. It would take much more data to accurately define and use a variable subgrade modulus for the foundation at Gull Lake Dam. However, the test data were adequate for an average value analysis of the piling foundation. A subgrade of $6000 \frac{\text{psi}}{\text{in.}}$ was used. If the piling layout was deemed adequate by this analysis, the whole dam could be considered adequate in stability.

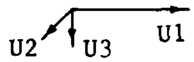
Pile Foundation Analysis Using In Situ Soil-Foundation Properties

46. A general, direct stiffness analysis for a three-dimensional pile foundation was used as presented by Saul (1968), which expanded the Hrennikoff (1950) method from two dimensions to three. The general solution using this stiffness analysis is presented below.

47. The forces on a single pile can be equated to the pile displacements by the expression

$$\{F\}_i = \{b\}_i\{X\}_i \tag{6}$$

The $\{b\}_i$ values are the individual pile stiffness-influence coefficients, called the elastic pile constants. The positive system is



48. The $\{b\}_i$ matrix for a three-dimensional system can be defined for the i^{th} pile as

$$\{b\}_i = \begin{bmatrix} b_{11} & 0 & 0 & 0 & b_{15} & 0 \\ 0 & b_{22} & 0 & b_{24} & 0 & 0 \\ 0 & 0 & b_{33} & 0 & 0 & 0 \\ 0 & b_{42} & 0 & b_{44} & 0 & 0 \\ b_{51} & 0 & 0 & 0 & b_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & b_{66} \end{bmatrix}$$

The elastic pile constants are defined as follows:

- b_{11} is the force required to displace the pile head a unit distance along the U_1-axis, FORCE/LENGTH.
- b_{22} is the force required to displace the pile head a unit distance along the U_2-axis, FORCE/LENGTH.
- b_{33} is the force required to displace the pile head a unit distance along the U_3-axis, FORCE/LENGTH.
- b_{44} is the moment required to displace the pile head a unit rotation around the U_1-axis, FORCE-LENGTH/RADIAN.
- b_{55} is the moment required to displace the pile head a unit rotation around the U_2-axis, FORCE-LENGTH/RADIAN.
- b_{66} is the torque required to displace the pile head a unit rotation around the U_3-axis, FORCE/RADIAN.
- b_{15} is the force along the U_1-axis caused by a unit rotation of the pile head around the U_2-axis, FORCE/RADIAN.
- $-b_{24}$ is the force along the U_2-axis caused by a unit rotation of the pile head around the U_1-axis, FORCE/RADIAN. (Note: The sign is negative.)
- b_{51} is the moment around the U_2-axis caused by a unit of displacement of the pile head along the U_1-axis, FORCE-LENGTH/LENGTH.

$-b_{42}$ is the moment around the U_1-axis caused by a unit displacement of the pile head along the U_2-axis, FORCE-LENGTH/LENGTH. (Note: The sign is negative.)

49. Pile i may be located in the foundation with axes through its origin parallel to the foundation axes. The foundation loads $\{Q\}$ and displacements $\{\Delta\}$ are located with respect to the foundation axes.

50. The forces $\{F\}_i$ due to the pile on the pile cap are in equilibrium with a set of forces $\{q\}_i$ at the coordinate center of the pile cap. Equilibrium yields:

$$\{q\}_i = \{c\}_i \{F'\}_i \tag{7}$$

in which $\{c\}_i$, the statics matrix for a three-dimensional system, is

$$\{c\}_i = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -u_3 & u_2 & 1 & 0 & 0 \\ u_3 & 0 & -u_1 & 0 & 1 & 0 \\ -u_2 & u_1 & 0 & 0 & 0 & 1 \end{bmatrix}$$

where

$u_1 = U_1$ coordinate of the pile, LENGTH.

$u_2 = U_2$ coordinate of the pile, LENGTH.

$u_3 = U_3$ coordinate of the pile, LENGTH.

Foundation stiffness analysis

51. If the piling cap is assumed rigid, then the deflection of the pile cap can be related to the deflection of the piling in the foundation axis coordinates by

$$\{x'\}_i = \{c\}_i^T \{\Delta\} \tag{8}$$

52. The foundation load $\{Q\}$ is distributed to each piling so that

$$\{Q\} = \sum_{i=1}^{n} \{q\}_i \tag{9}$$

where n = number of piles. The relationships between the foundation load and the pile cap deflections are

$$\{Q\} = \{S\}\{\Delta\} \tag{10}$$

in which {S} is the stiffness-influence coefficients matrix for the foundation as a whole. The {S} matrix is found by introducing the contribution of each individual pile toward the stiffness of the pile cap. This yields

$$\{q\}_i = \{S'\}_i\{\Delta\} \tag{11}$$

in which

$$\{S'\}_i = \{c\}_i\{a\}_i\{b\}_i\{a\}_i^T\{c\}_i^T \tag{12}$$

and finally

$$\{S\} = \sum_{i=1}^{n} \{S'\}_i \tag{13}$$

where {a} is the tranformation matrix of force and displacement of the pile (rotated and/or battered) axis to the foundation axis. Once the stiffness matrix is known for the total foundation, the problem is essentially solved and only requires back substitution to find the distribution of loads to the individual piling. It can be noted that the foundation stiffness matrix {S} is dependent of the external loads.

Loads and displacements

53. The displacements of the pile cap can be found by inverting the foundation stiffness matrix {S} and multiplying it by the external load matrix {Q} or,

$$\{\Delta\} = \{s\}^{-1}\{Q\} \tag{14}$$

Once the foundation deflections are known, the deflection of pile i about its own axes can be found by

$$\{x\}_i = \{a\}_i^T\{c\}_i^T\{\Delta\} \qquad (15)$$

Finally, the forces allotted to each pile about its axes can be found from Equation 6 where

$$\{F\}_i = \{b\}_i\{x\}_i \qquad (16)$$

Forces and Deflections of Individual Piles

54. The approach followed in obtaining the forces and deflections on the individual pile is as follows. The modulus of reaction, the material properties of the pile, and the pile length were used to determine the pile head-stiffness matrix for a single pile, assuming a linear elastic pile-soil system. This pile head-stiffness matrix was obtained by using a finite element computer code (Marlin, Jones, and Radhakrishnan, in preparation), which is a one-dimensional finite element analysis of a beam on an elastic foundation.

55. The pile head-stiffness matrix was then used as input in another computer program that uses the direct stiffness analysis to obtain the forces and deflections of the piles. A beam on an elastic foundation analysis was performed and the pressures, moments, and deflections along the length of the most critically loaded pile were determined.

56. The analysis assumes that the top of the pile is pinned to the base of the monolith, and the monolith base is rigid. These assumptions were adequate for the dam construction at Gull Lake.

57. The results of the three-dimensional pile foundation analysis are presented in Table 9. The pressures, moments, and deflections along the length of the pile for the most critical load case (normal operation with ice) are presented in Figure 32. The stresses and shears were below the allowables for the piling. The stresses compare to the allowables as follows:

Maximum compressive stress = 637 < 1100 psi allowable,

Tensile stress = 205 psi < 775 psi allowable, and

Maximum shear stress = 68.5 psi $<$ 75 psi allowable.

The compressive, tensile, and shear loads per pile are not excessive. The moment in the piling is not excessive. The conventional stability analysis obtains pile loads which compare well with those obtained by the direct stiffness analysis.

58. The deflections of the piling were less than one quarter inch for all load cases and were acceptable. Since there are no noticeable vertical deflections of the structure and due to the adequacy of the foundation, it is not expected that vertical deflections will cause any loss of reservoir pool in the future. The vertical deflections were computed only as axial deflections of the piles $(\frac{PL}{AE})$ to save time and expense. The pile foundation at Gull Lake Dam is adequate in stability.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

59. The soil-piling system, which supports the monoliths at Gull Lake Dam, was adequate. The foundation material is sand with reliable in situ supporting capabilities. The pilings have been continuously submerged; therefore, they will be nondeteriorated and adequate. During the drilling program for the Upper Mississippi Headwater Structures, pieces of piling were obtained (at various locations) that support this conclusion.

Recommendations

60. The concrete is generally of good quality with minor surface freezing-and-thawing deterioration. It is recommended that within the next five years an economical method be used to rehabilitate the deteriorated surface concrete in order that water is not allowed to enter cracks and accelerate deterioration. It is expected that this structure will be adequate for many more years of service with routine maintenance and some inexpensive rehabilitation of the deteriorated surface concrete.

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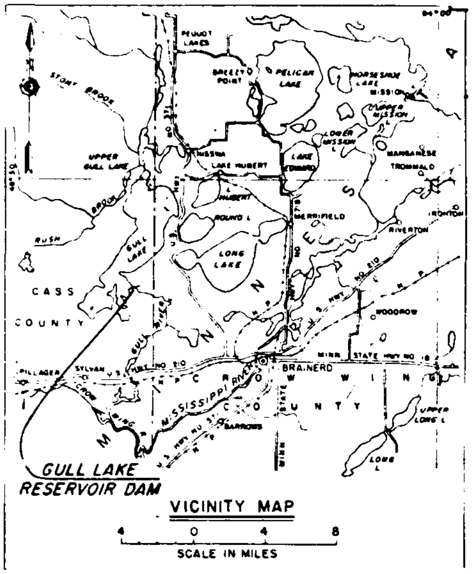
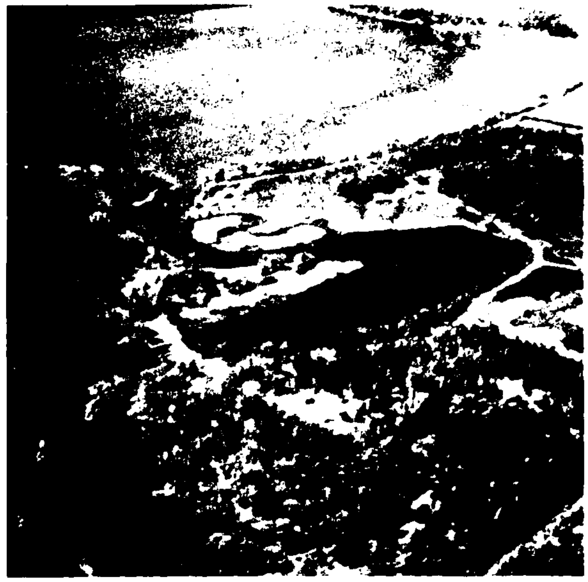
U. S. Army Engineer District, St. Paul. 1973. "Reservoirs at Headwaters, Mississippi River, Minnesota, Gull Lake Dam, Periodic Inspection Report No. 1," Corps of Engineers, St. Paul, Minn.

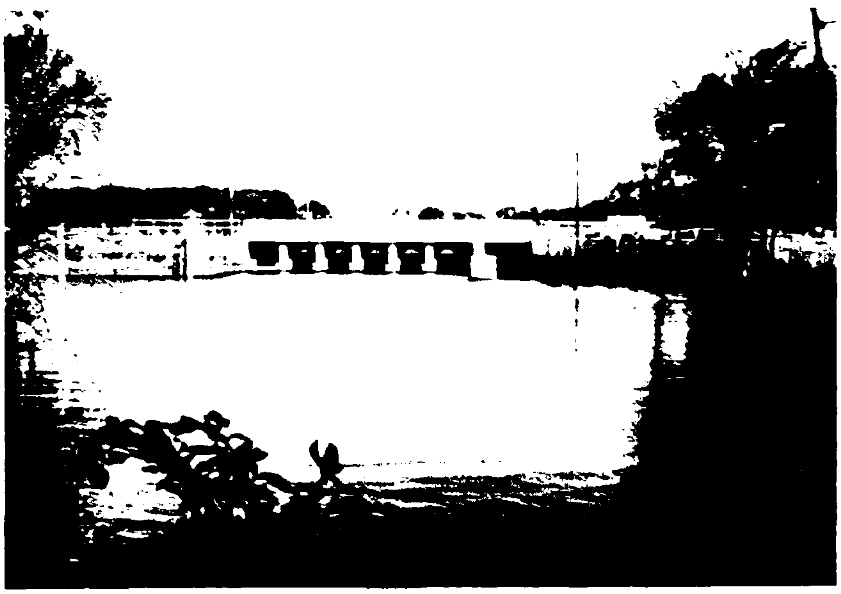
U. S. Army Engineer District, St. Paul. 1977. "Mississippi River, Headwaters Reservoirs, Master Plan for Public Use Development and Resource Management," Corps of Engineers, St. Paul, Minn.



a. Vicinity map



b. Aerial site photo



c. General downstream view of Gull Lake Dam

Figure 1. Gull Lake area map and photos

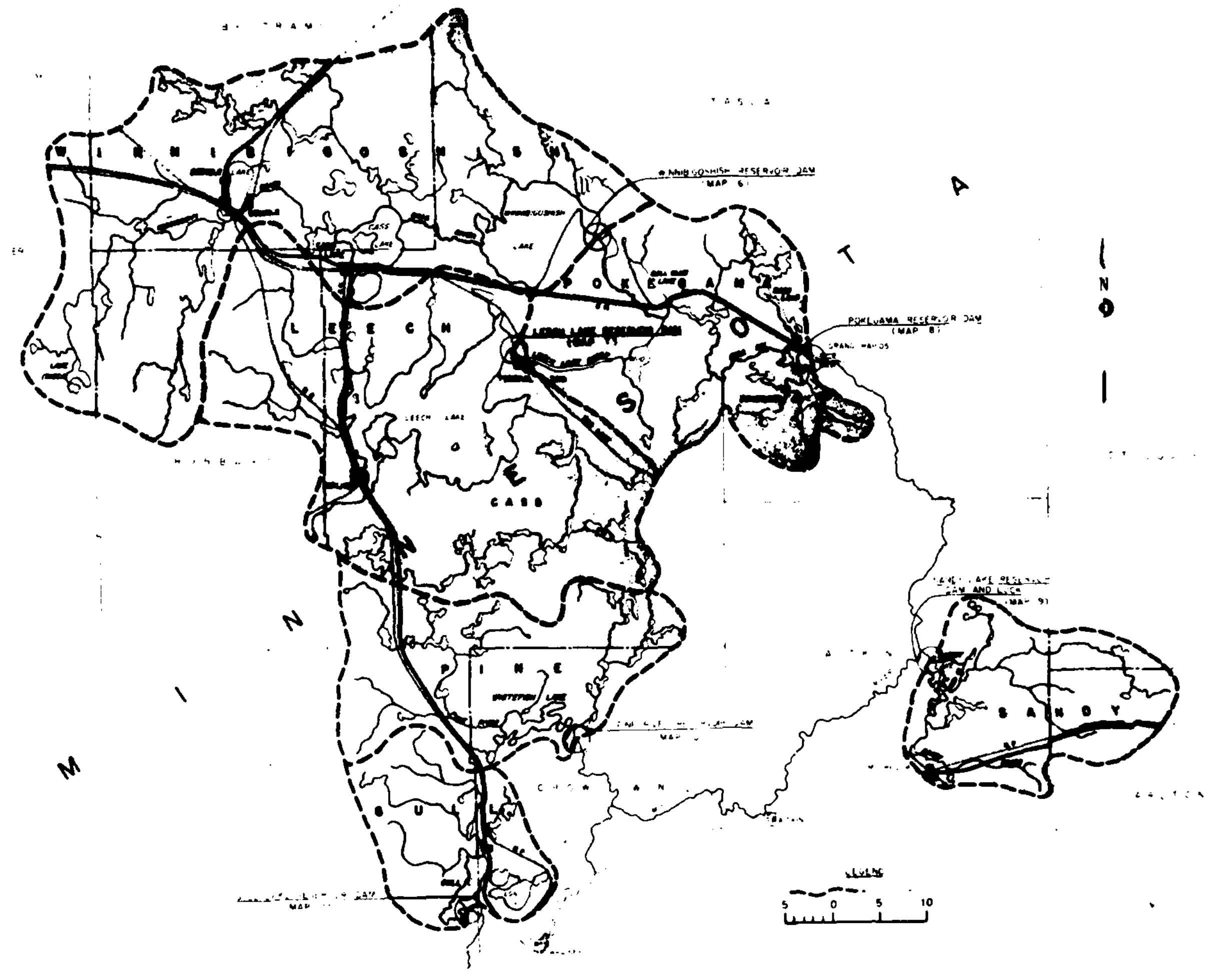
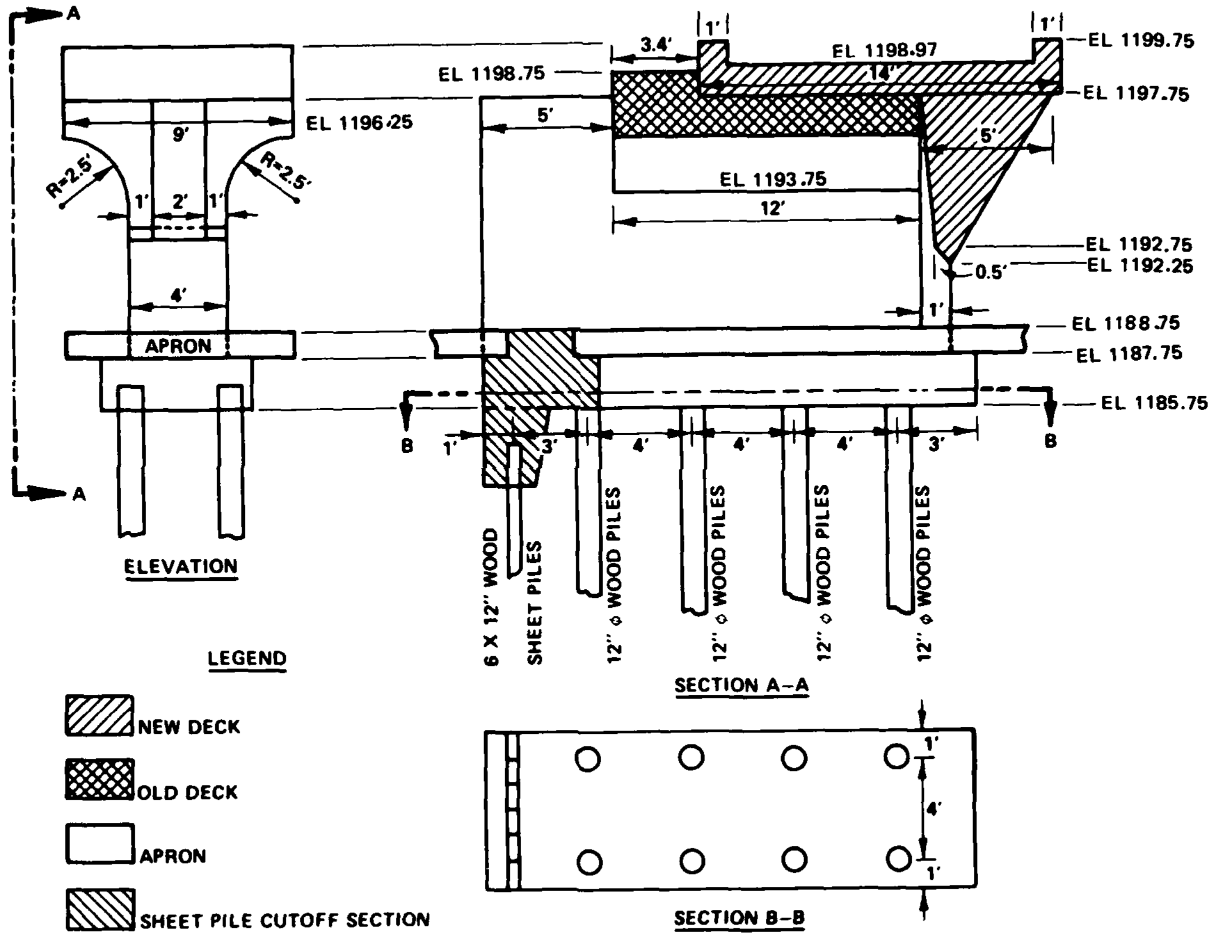


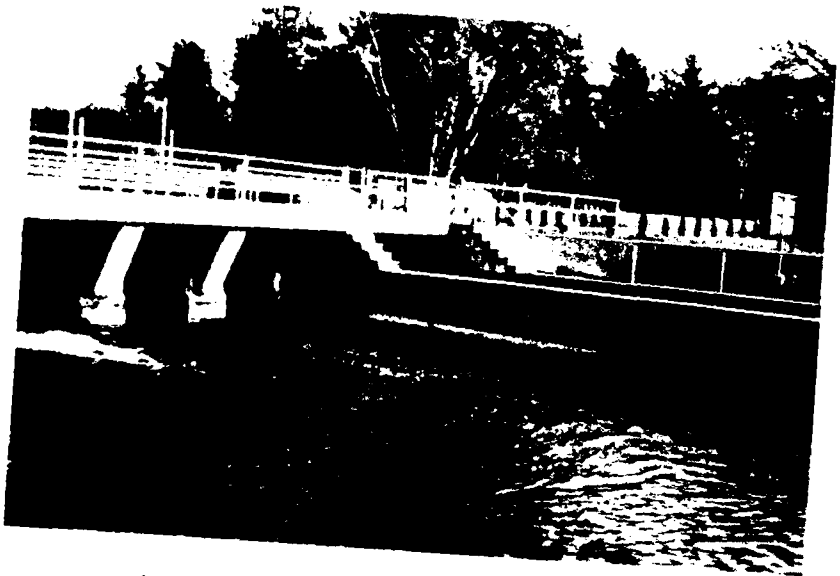
Figure 2. Project map of Mississippi River headwaters reservoirs



UPSTREAM VIEW AND SECTIONS OF DAM

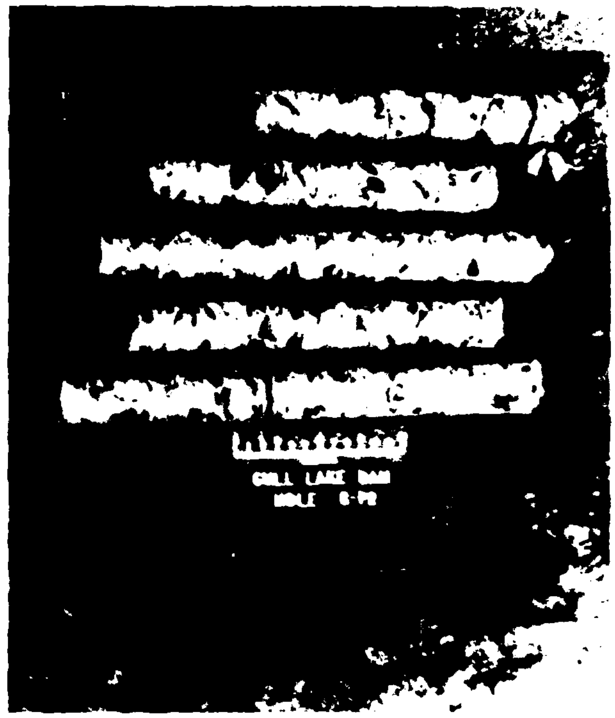
Figure 3. General construction of Gull Lake Dam

a. Upstream view of dam

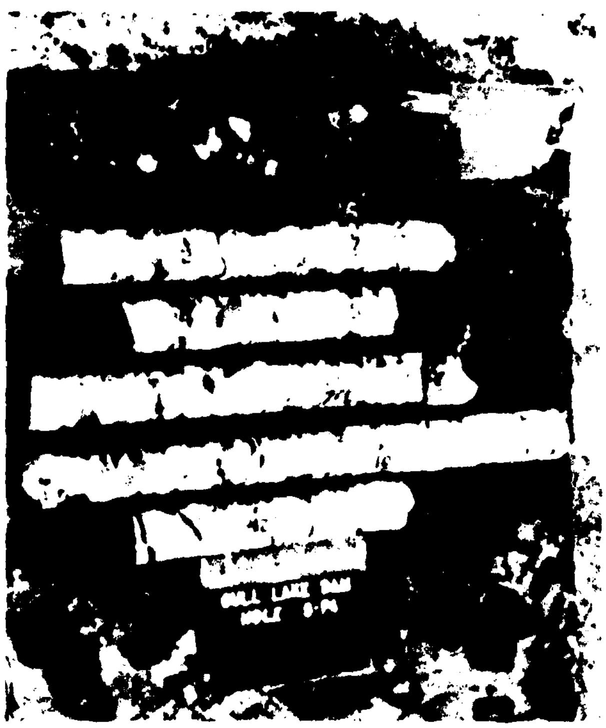


b. Side view of dam and log sluice

Figure 4. Views of Gull Lake Dam



a. Cores from G-P2 boring



b. Cores from G-P4 boring

Figure 5. Gull Lake test cores

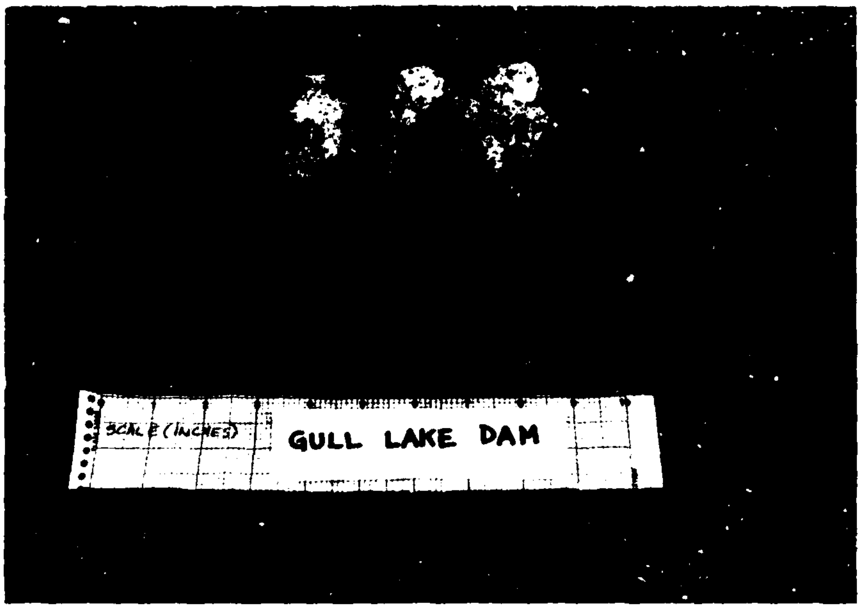
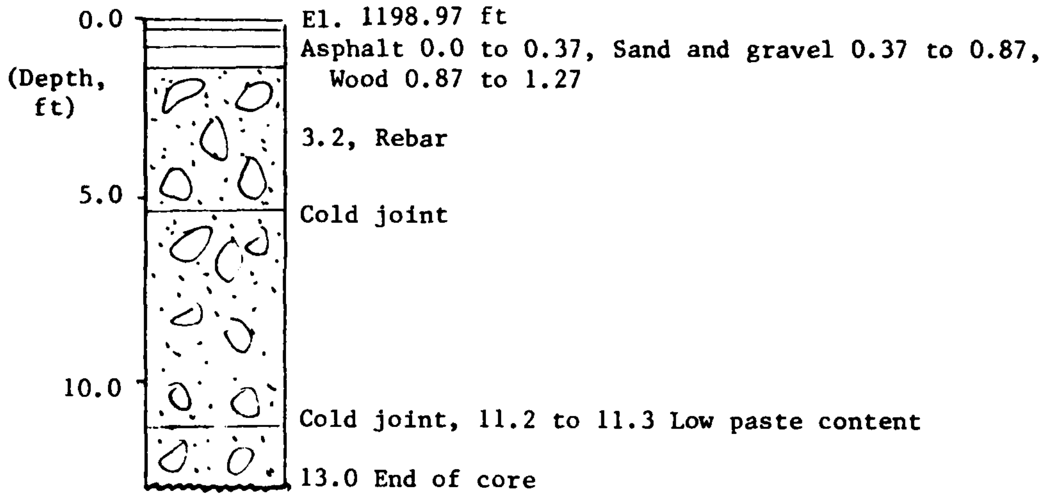


Figure 6. Core and cut section of G-P4 core from Gull Lake Dam



2-in. maximum size crushed aggregate composed of granite, granite gneiss, and dark, fine-grained igneous rock particles.

Some alkali-silica reaction gel was found coating aggregate surfaces.

Ettringite needles found in voids and coating aggregate surfaces.

Non air-entrained concrete.

Fine aggregate was natural sand.

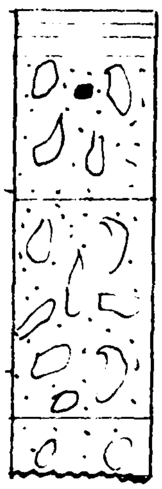
Figure 7. Core log, hole G-P2, Gull Lake Dam

0.0

(Depth, ft)

5.0

10.0



El. 1198.97 ft

Asphalt 0.0 to 0.38, Sand and gravel 0.38 to 0.90,
Wood 0.90 to 1.30

Rebar

Cold joint

Cold joint

13.0 End of core

2-in. maximum size crushed aggregate composed of granite, granite gneiss, and dark, fine-grained igneous rock particles.
Some alkali-silica reaction gel was found coating aggregate surfaces.
Ettringite needles found in voids and coating aggregate surfaces.
Fine aggregate was natural sand.
Non air-entrained concrete.

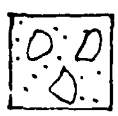
 Concrete

Figure 8. Core log, hole G-P4, Gull Lake Dam

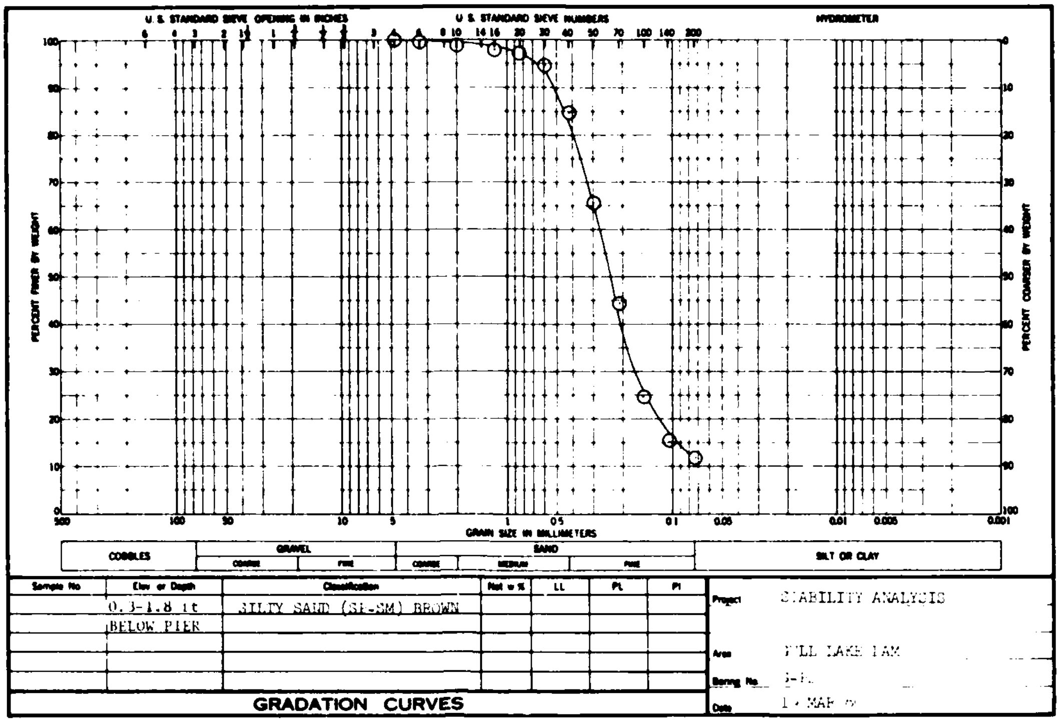


Figure 9. Foundation soil sieve analysis and classification, G-P2 (depth 0.3-1.8 ft)

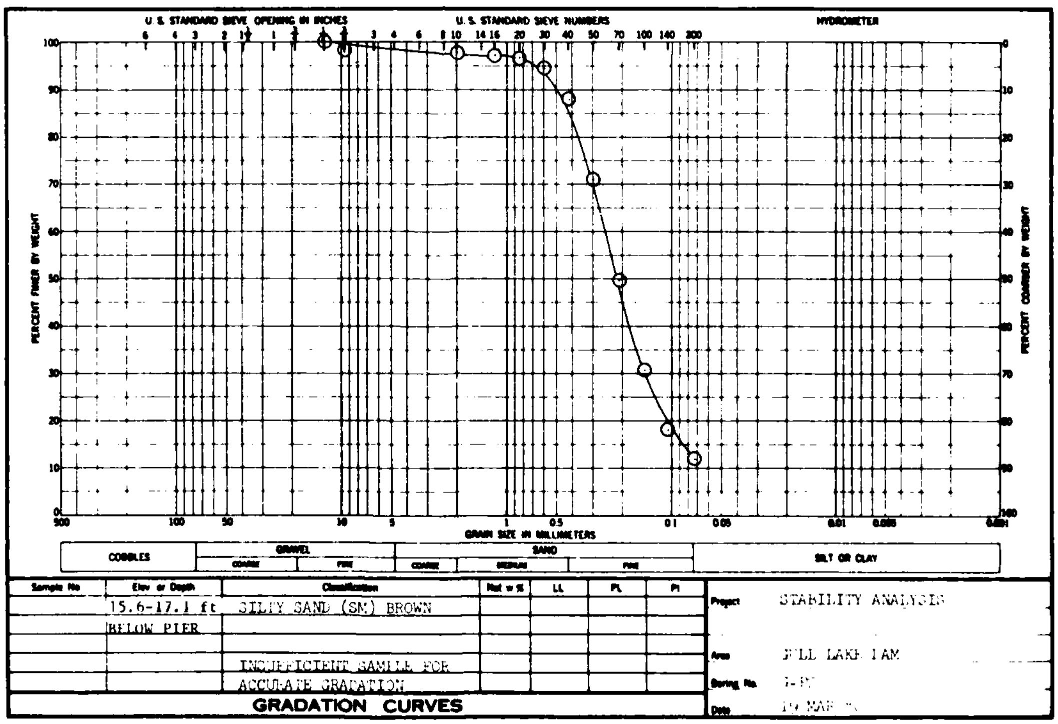


Figure 10. Foundation soil sieve analysis and classification, G-P2 (depth 15.6-17.1 ft)

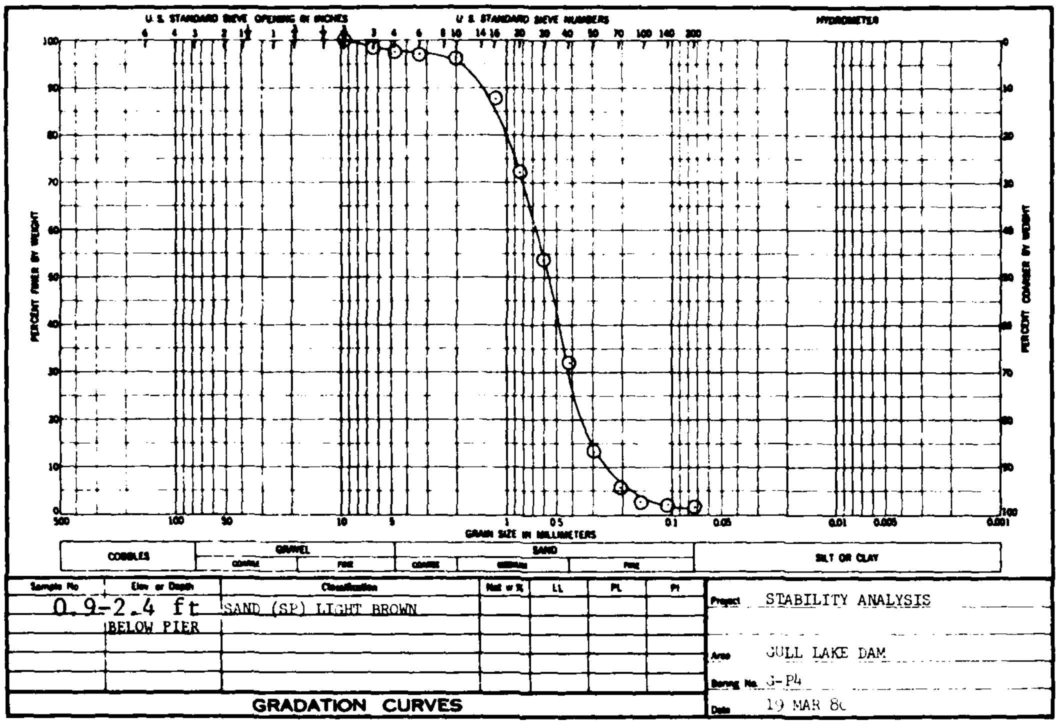


Figure 11. Foundation soil sieve analysis and classification, G-P4 (depth 0.9-2.4 ft)

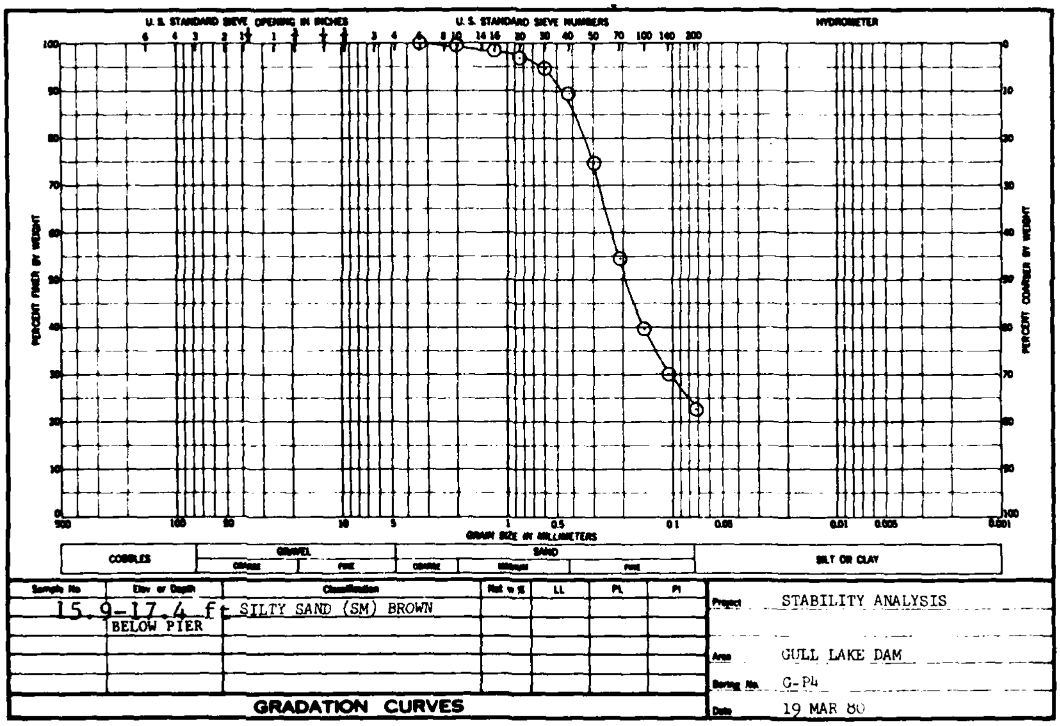


Figure 12. Foundation soil sieve analysis and classification, G-P4 (depth 15.9-17.4 ft)

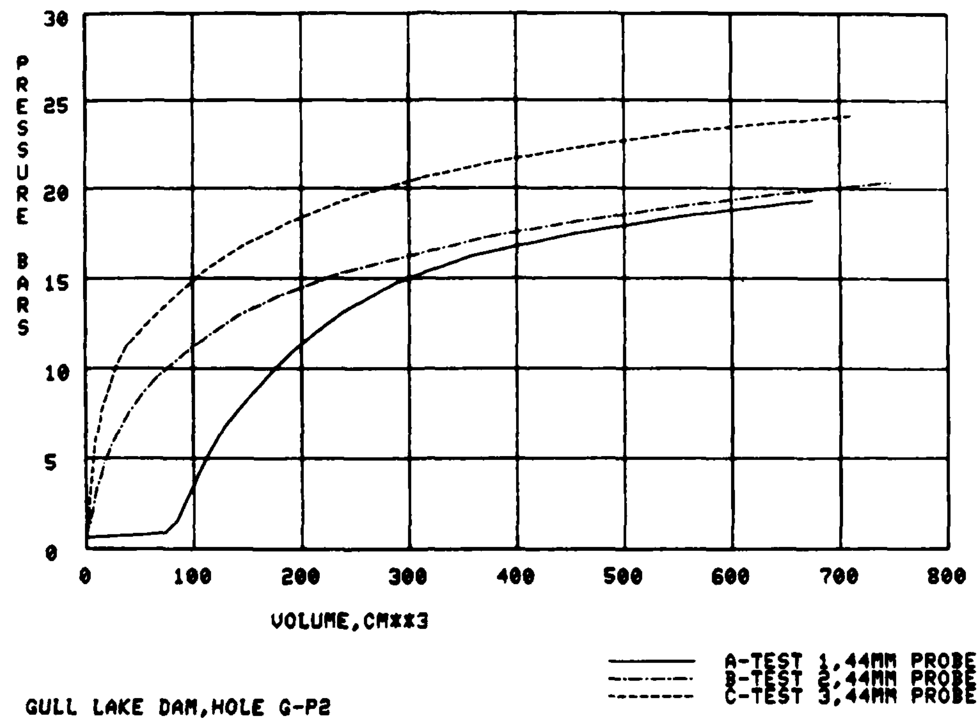


Figure 13. Pressure versus volume, Gull Lake Dam, hole G-P2

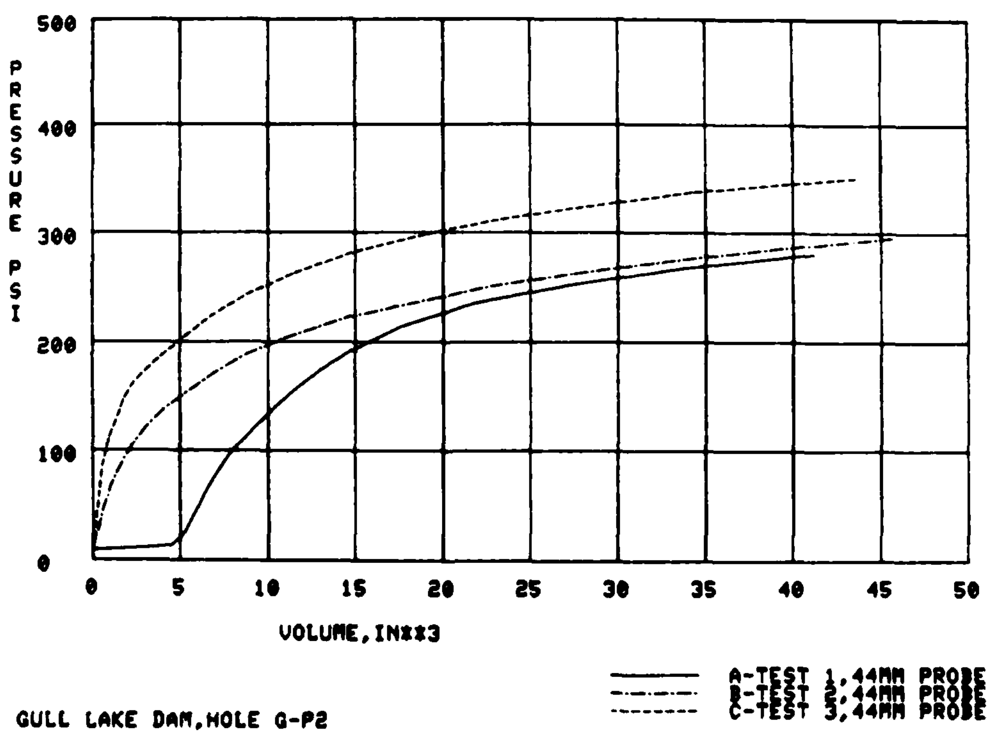


Figure 14. Pressure versus volume, Gull Lake Dam, hole G-P2

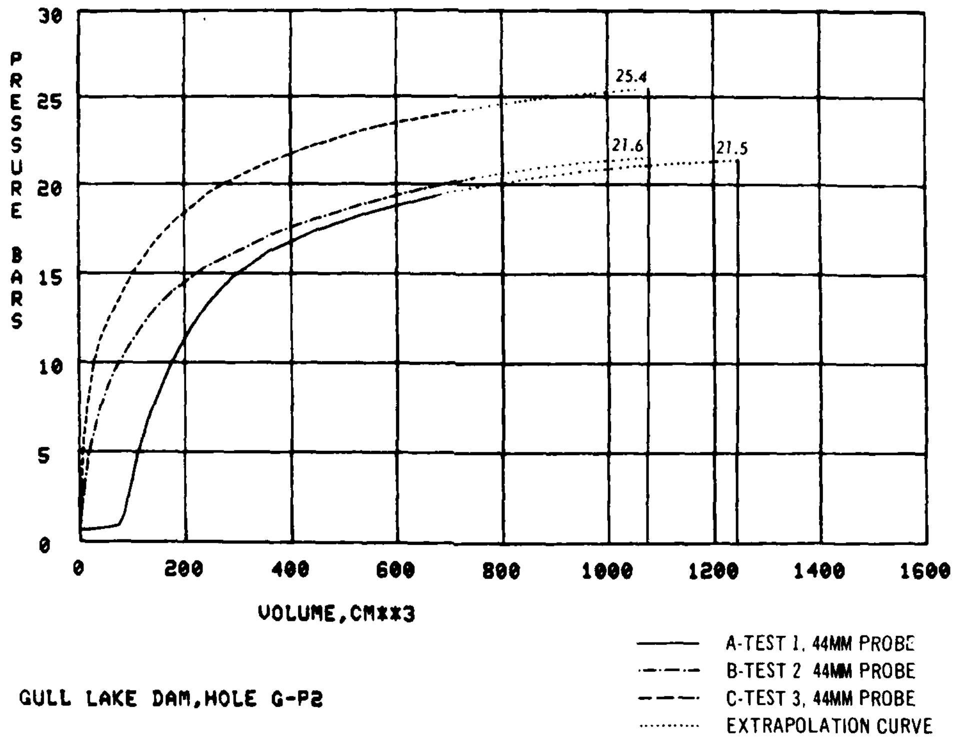


Figure 15. Pressure versus volume, curves extrapolated to obtain limit pressures, Gull Lake Dam, hole G-P2

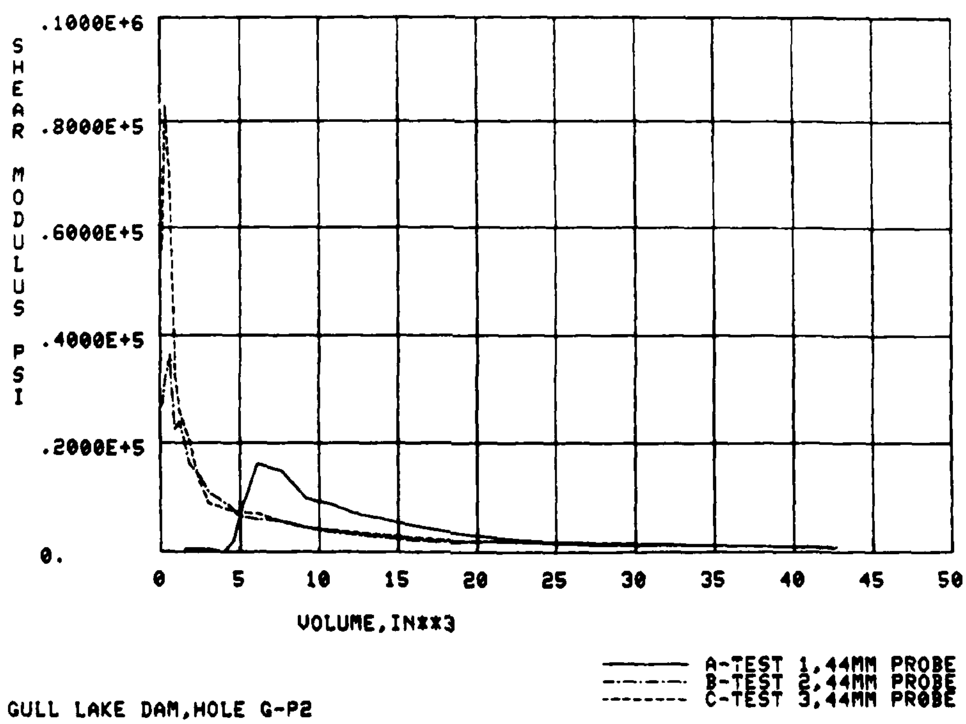


Figure 16. Shear modulus, Gull Lake Dam, hole G-P2

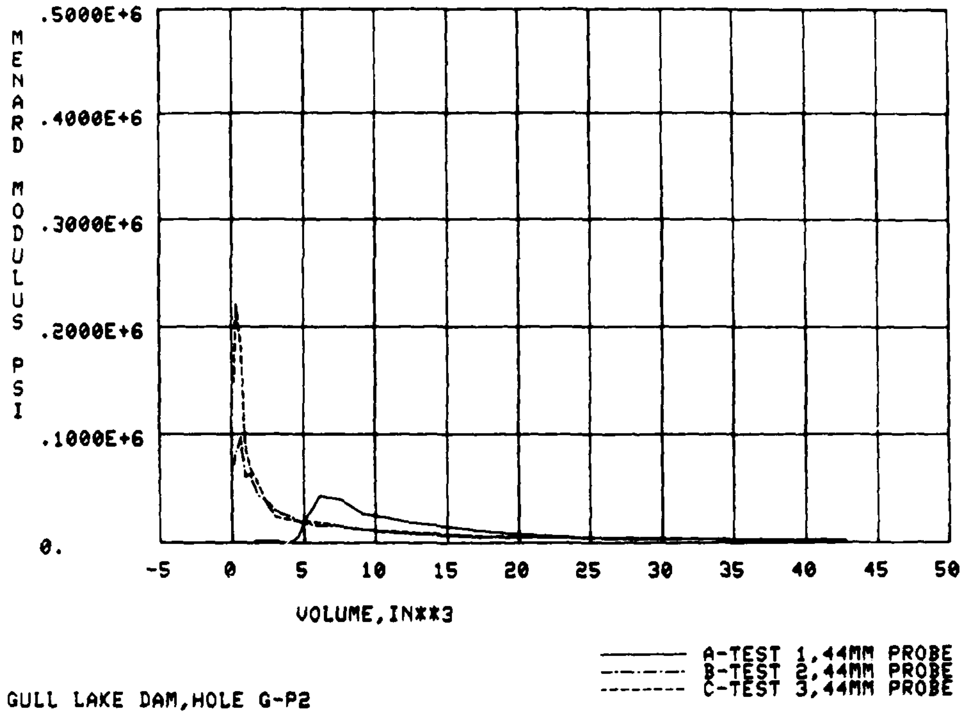


Figure 17. Ménard modulus, Gull Lake Dam, hole G-P2

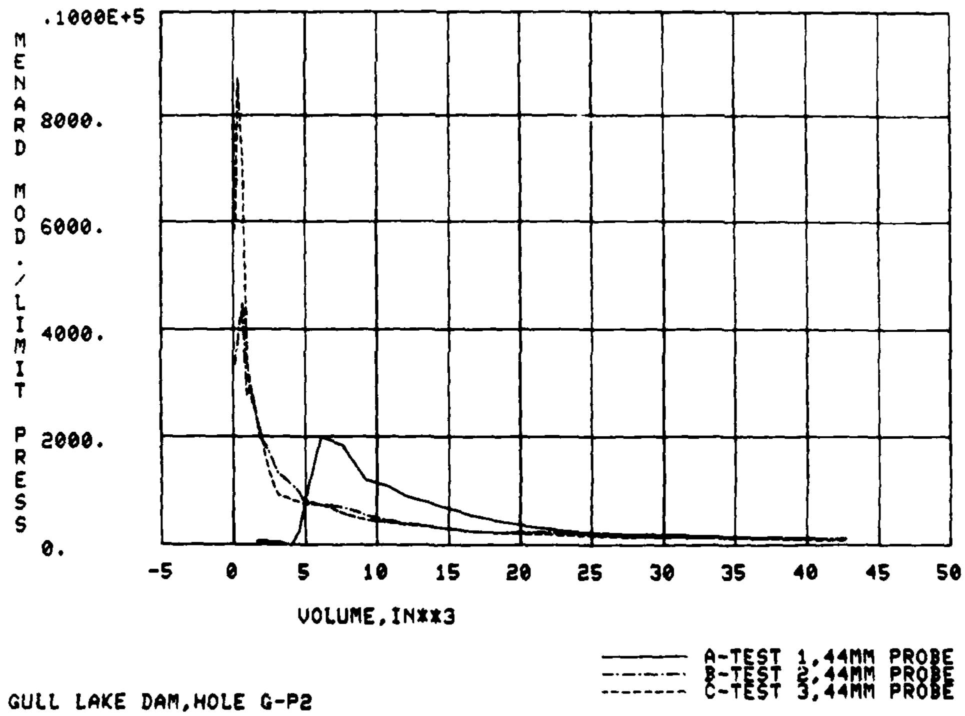


Figure 18. Ménard modulus divided by limit pressure, Gull Lake Dam, hole G-P2

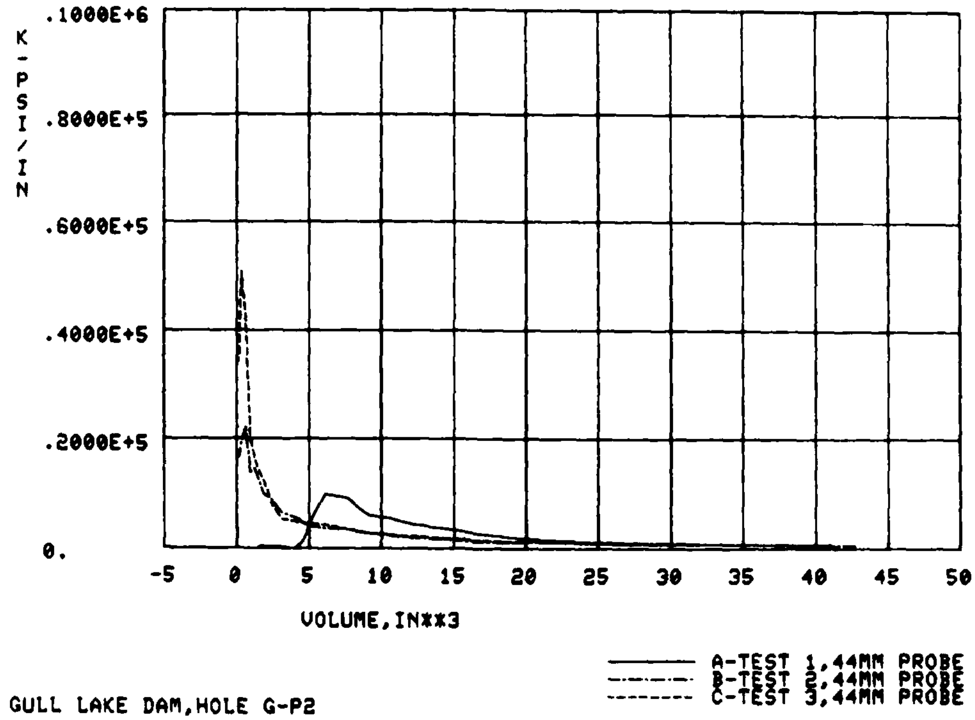


Figure 19. Modulus of subgrade reaction, Gull Lake Dam, hole G-P2

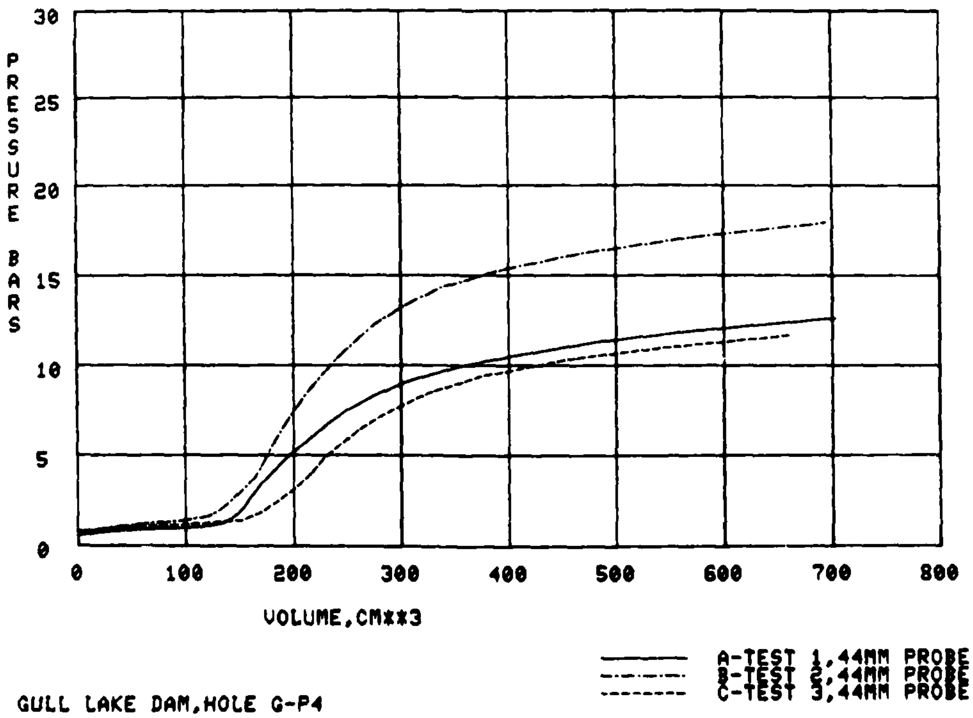


Figure 20. Pressure versus volume (metric units), Gull Lake Dam, hole G-P4

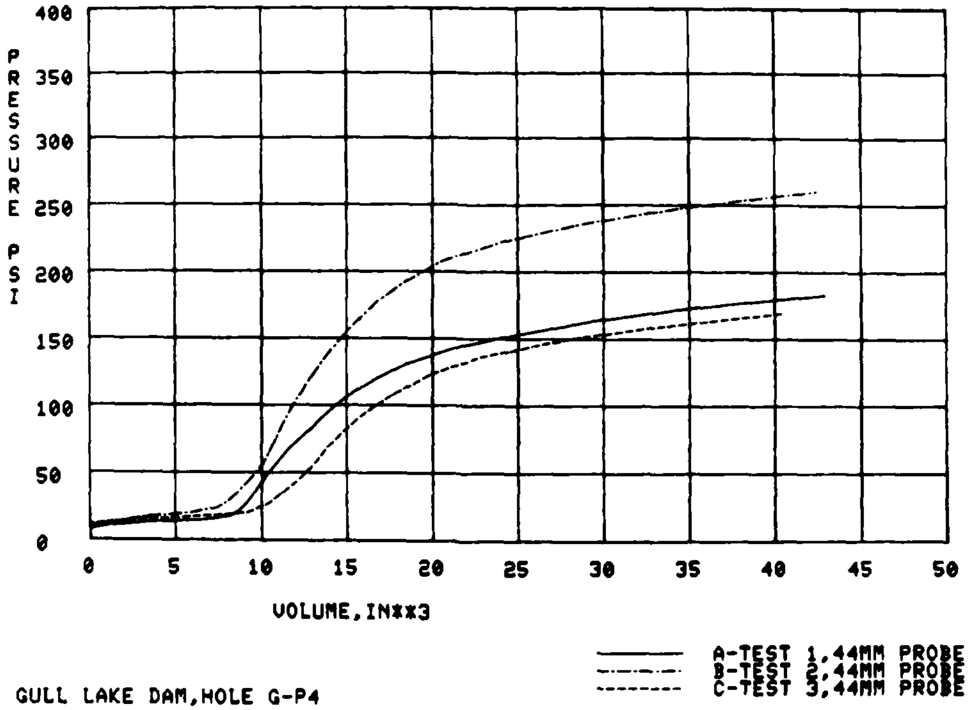


Figure 21. Pressure versus volume, Gull Lake Dam, hole G-P4

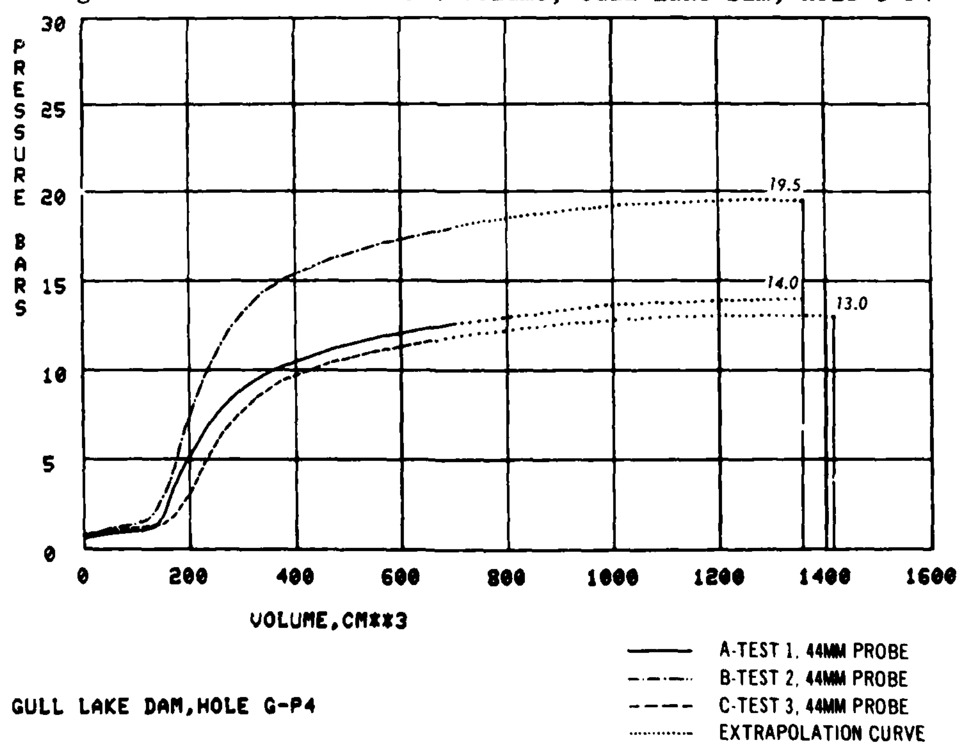


Figure 22. Pressure versus volume, curves extrapolated to obtain limit pressures, Gull Lake Dam, hole G-P4

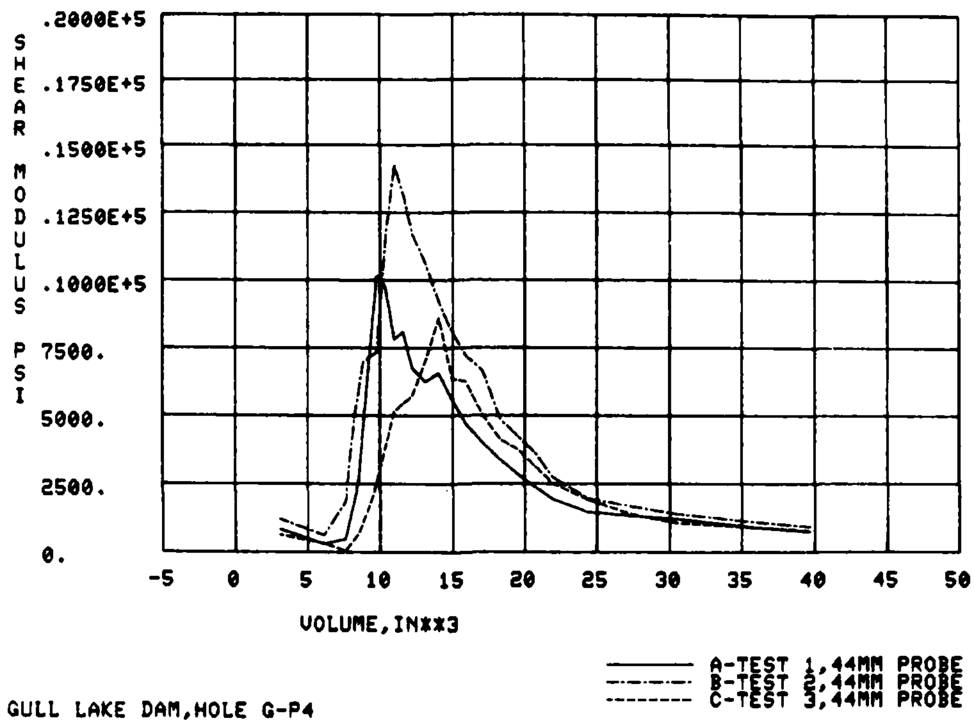


Figure 23. Shear modulus, Gull Lake Dam, hole G-P4

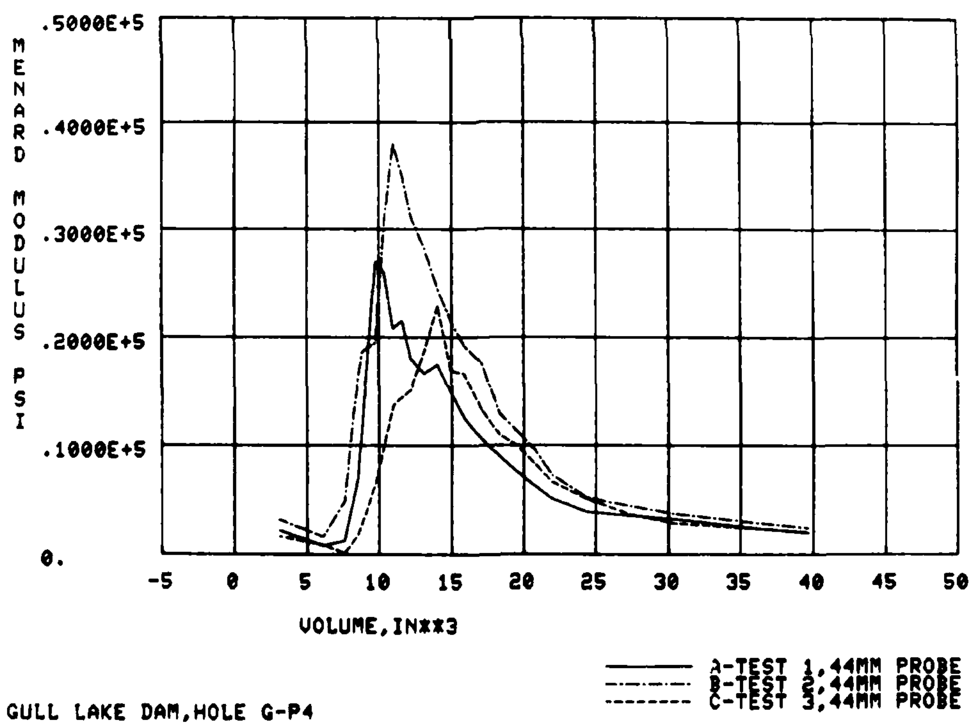


Figure 24. Ménard modulus, Gull Lake Dam, hole G-P4

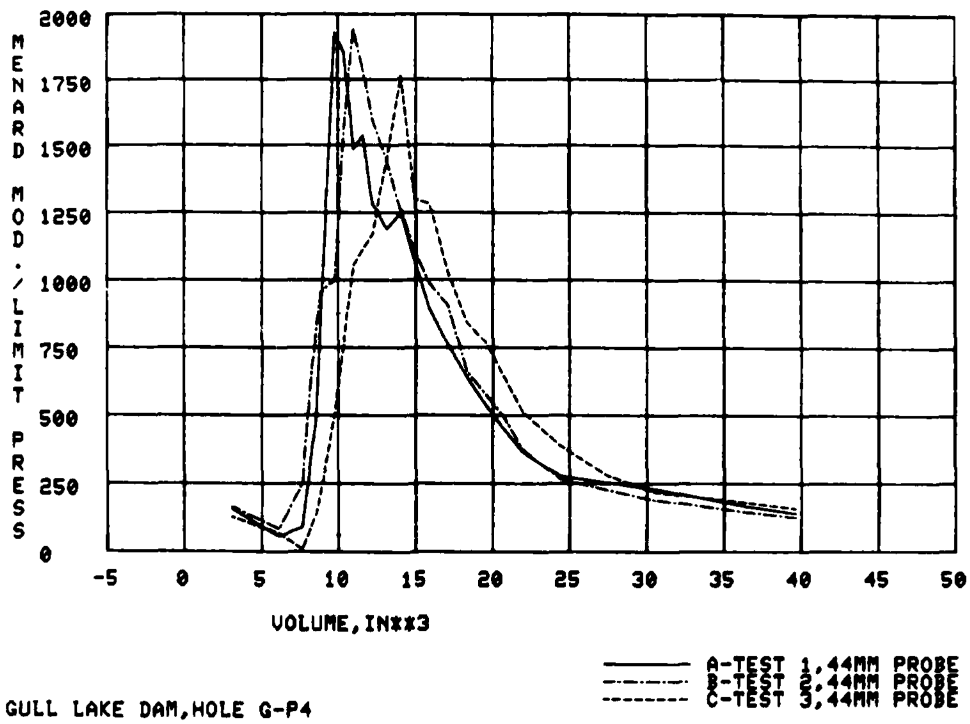


Figure 25. Ménard modulus divided by limit pressure, Gull Lake Dam, hole G-P4

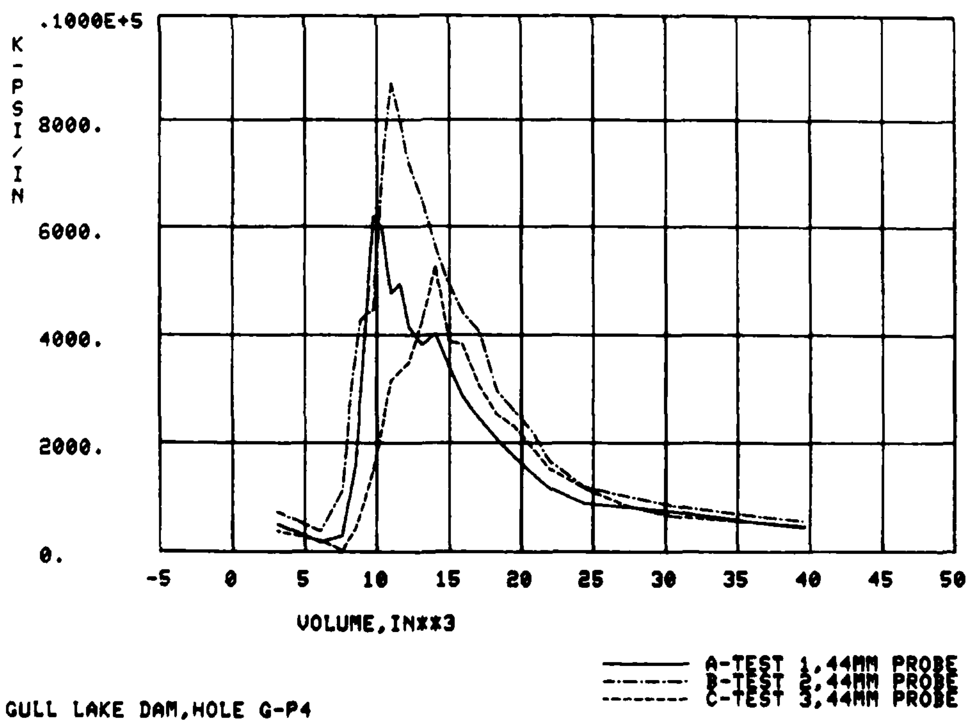


Figure 26. Modulus of subgrade reaction, Gull Lake Dam, hole G-P4

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$W_{Roadway\ and\ pier}$	(2)(1)(0.78)(9)(0.15)	2.11		3.6		7.6
	(1.22)(14)(9)(0.15)	23.06		3.6		83.0
	(3.4)(1)(9)(0.15)	4.59		12.3		56.5
	(12)(1.5)(9)(0.15)	24.30		8.0		194.4
	(0.35)	0.35		3.6		1.3
	(2)(4.5)(5.5)(1/2)(0.15)	3.71		0		0
	(4)(7.5)(17.5)(0.15)	78.75		10.25		807.2
	$[(5)(2.5) - \frac{(3.14)(2.5)^2}{2}](12.5)(0.15)$	5.04		7.75		39.1
	(5)(1.5)(9)(0.15)	10.13		16.5		167.2
	(19)(1)(9)(0.15)	25.65		9.5		243.7
	(6)(2)(14.5)(0.15)	26.10		7.25		189.2
	(15.75)(9)(0.15)	21.26		17.23		366.3
		225.05				2,155.5
$W_{Water\ apron}$	(1.5)(5)(5)(0.0625)	2.34		18.25		42.7
$P_{Headwater}$	$-(0.0625)(1193.75-1182.75)^2(1/2)(9)$		-34.03		0.67	-22.8
$P_{Tailwater}$	$+(0.0625)(1187.75-1185.75)^2(1/2)(9)(0.6)$		- 0.68		-0.67	0.5
$Uplift_{pier}$	-[11 - 6 - 3](0.0625)(19)(6)	-14.25		9.5		-135.4
	$-[11 - \frac{(6)(35.04)}{(51.54)} - 3.0 - [11 - 6 - 3.0]](0.0625)(16.5)(6)(1/2)$	-5.94		11.0		-65.3
	$-[11 - \frac{(4.55)(35.04)}{(51.54)} - 3 - [11 - 6 - 3.0]](0.0625)(2.5)(6)$	-1.80		17.75		-32.0
	$-[11 - \frac{(6)(32)}{(51.54)} - [11 - \frac{(6)(35.04)}{(51.54)} - 3.01(0.0625)(0.5)(6)(1/2)$	-0.31		16.83		-5.2
	$-[11 - \frac{(6)(32)}{(51.54)} - [11 - \frac{(6)(35.05)}{(51.54)} - 3.01(0.0625)(2)(6)$	-2.52		18.0		-45.4
	$-[11 - \frac{(6)}{(51.54)}(30) - [11 - \frac{(6)}{(51.54)}(32)]$ (0.0625)(2)(6)(1/2)	-0.04		18.33		-1.6
	$-[11 - \frac{(6)}{(51.54)}(1) - [11 - \frac{(6)}{(51.54)}(32)]$ (0.0625)(1)(6)	1.35		18.5		-25.0
		-26.26				-309.9

Figure 27. Stability, Gull Lake Dam, normal operation (continued).

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$Uplift_{Apron}$	(0.13)	-5.99		14.27		-85.5
$Uplift_{(Horizontal)}$	(0.25)(3)(9)		6.75		-1.5	-10.1
	(0.46 - 0.25)(3)(9)(1/2)		2.84		-2.0	-5.7
			9.59			-15.8
	Total	195.14	-23.76			1764.7

$$e = \frac{1764.7}{195.14} = 9.04 \text{ ft}$$

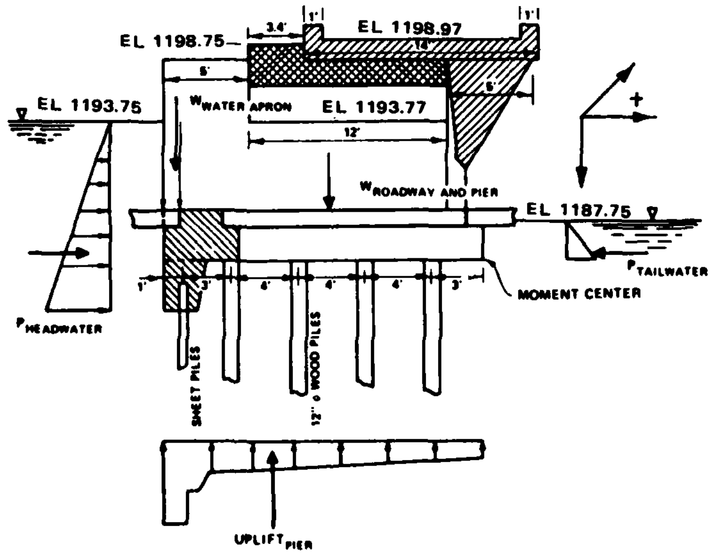


Figure 27. (concluded).

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$W_{Roadway\ and\ pier}$		225.05				2,155.5
$W_{Water\ apron}$		2.34				42.7
$P_{Headwater}$	From Normal Operation Case		-34.03		0.67	-22.8
$P_{Tailwater}$			0.68		0.67	0.5
$Uplift_{Pier}$		-26.26	9.59			-325.7
$Uplift_{Apron}$		-5.99				-85.5
Truck Load (H15-44)		24		2.6		62.4
		219.14	-23.76			1,784.4

$$e = \frac{1784.4}{219.14} = 8.14 \text{ ft}$$

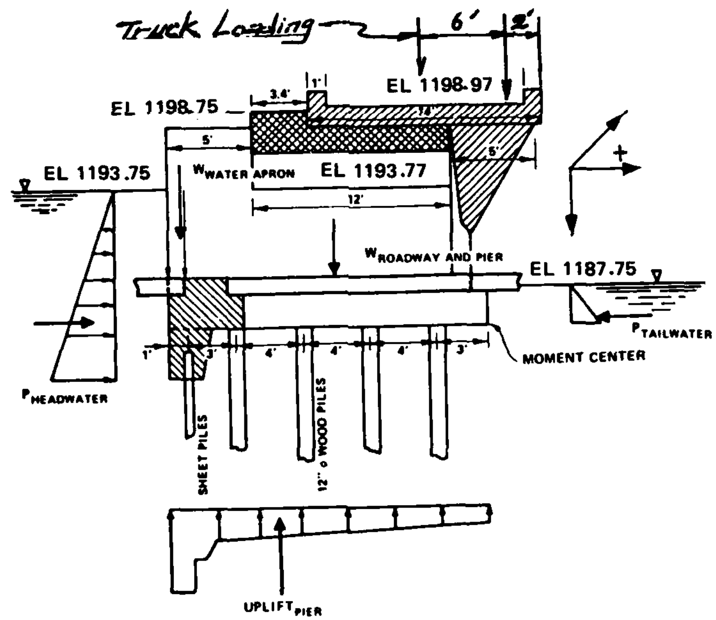


Figure 28. Stability, Gull Lake Dam, normal operation with truck loading (H15-44)

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$W_{Roadway\ and\ pier}$		225.05				2,155.5
$W_{Water\ apron}$		2.23				42.7
$P_{Headwater}$	From Normal Operation Case		-34.03			-22.8
$P_{Tailwater}$			0.68		0.73	0.5
$Uplift_{Pier}$		-26.26	9.59			-325.7
$Uplift_{Apron}$		-5.99				-85.5
Earthquake						
Pe_1	(0.025)(225.05)		-5.63		6.60	-37.2
Pe_2	$(2/3)(51)(0.025)(5)^2(\frac{1}{1000})(9)$		-0.19		5.00	-1.0
		195.14	-29.58			1,726.5

$$e = \frac{1726.5}{195.14} = 8.85 \text{ ft}$$

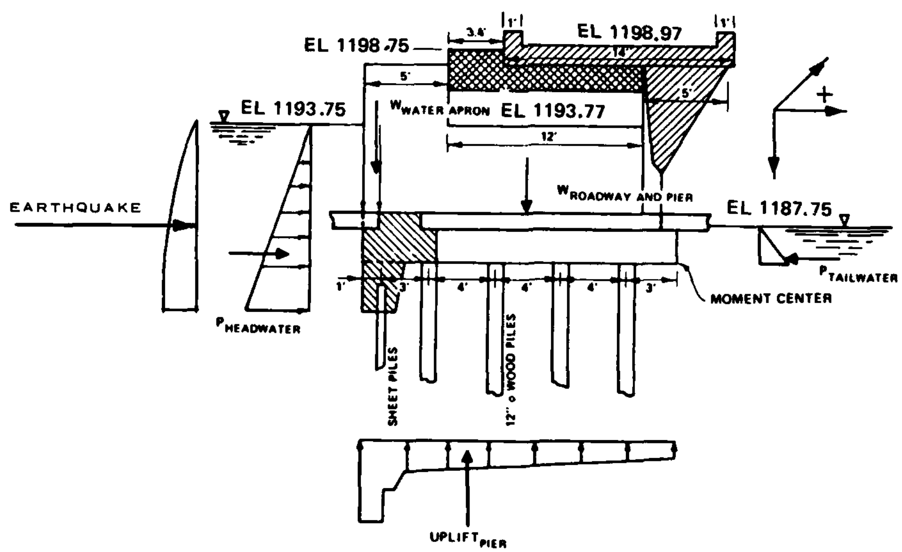


Figure 29. Stability, Gull Lake Dam, normal operation with earthquake

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$W_{Roadway\ and\ pier}$	From Normal Operation Case	225.05				2,155.5
$W_{Water\ apron}$		2.34				42.7
$P_{Headwater}$			-34.03			-22.8
$P_{Tailwater}$			0.68			0.5
$Uplift_{Pier}$		-26.26	9.59			-325.7
$Uplift_{Apron}$		-5.99				-85.5
Ice	(5)(9)				7.5	-377.5
		195.14	-68.76			1,427.2

$$e = \frac{1427.2}{195.14} = 7.31 \text{ ft}$$

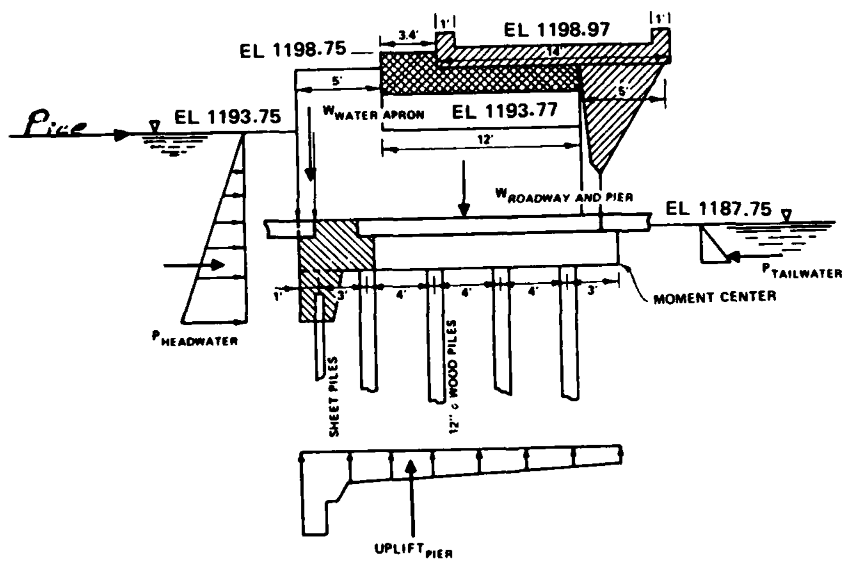


Figure 30. Stability, Gull Lake Dam, normal operation with ice

Item	Factors	F_V z-direction kips	F_H y-direction kips	Arm_y	Arm_z	M_x^+
$W_{Roadway\ and\ pier}$	(From Normal Operation Case)	225.05				2,155.5
W_{Water}	(1.5)(7)(5)(0.0625)	3.29		18.25		59.9
	(.0625)(1190.75 - 1188.75)(17.5)(5)	10.94		8.75		95.7
$P_{Headwater}$	$-(0.0625)(1195.75 - 1182.75)^2(1/2)(9)$		-47.53		1.33	-63.2
$P_{Tailwater}$	$+(0.0625)(1190.75 - 1185.75)^2(1/2)(9)(0.6)$		4.22		1.67	7.0
$Uplift_{Pier}$	(Computed in similar manner as for Normal Operation Case)	-46.06	13.92			-513.7
$Uplift_{Apron}$		-13.49				-141.0
	Total	179.72	-29.39			1600.2

$$e = \frac{1600.2}{179.72} = 8.90 \text{ ft}$$

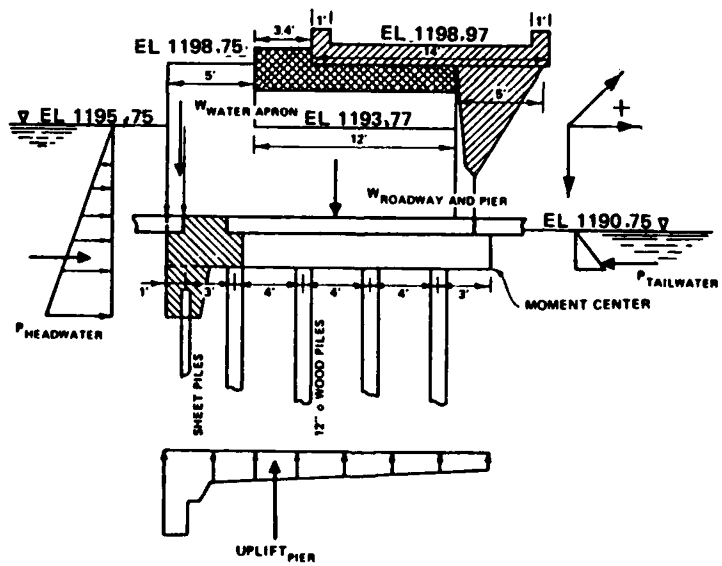


Figure 31. Stability, Gull Lake Dam, high-water condition

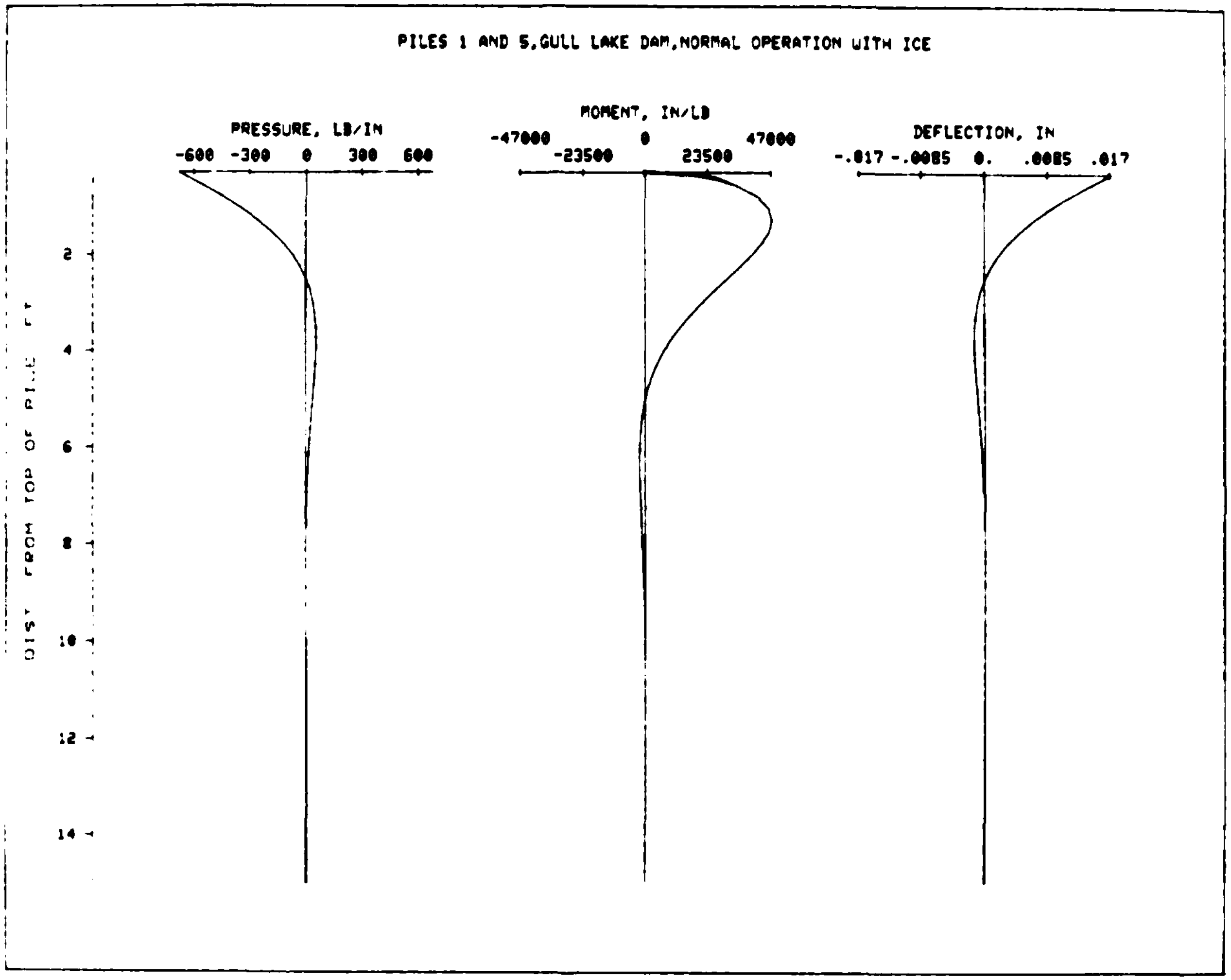


Figure 32. Pressures, moments, and deflections along length of pile for most case loading (normal operations plus ice).

Table 1

General Reservoir Data, Gull Lake Dam and Reservoir

Location in miles above Ohio River	1006.4
Located on river	Gull
Drainage area in square miles	287
Origin 1 operating limits	
Stage	1.0 to 7.0 ft
Storage in 1000 acre-ft	71.4
Present operating limits	
Stage	5.0 to 7.0 ft
Storage in 1000 acre-ft	26.2
Ordinary operating limits	
Stage	5.0 to 7.0 ft
Storage in 1000 acre-ft	26.2
Flowage rights to stage	+11 ft
Maximum stage of record (1914)	7.3 ft
Number of times upper operating limit exceeded	8
Number of times flowage limit exceeded	0
Maximum stage in 1950	7.26 ft
Maximum discharge in second-feet of record	1140
and year	1938
Elevation of gage zero (U.S.E. datum)	1190.00
Elevation of gage zero (msl 1912 adj.)	1188.14
(msl 1929 adj.)	1187.75
Year of first operation	1912
Normal spring stage drawdown	5.0 ft
Normal summer range	6.0 to 6.25 ft
Desirable bridge clearance (9.0 ft) above reservoir stage of	7.0 ft

Table 2

General Dam Data, Gull Lake Dam

Dam	
Type	Concrete curtain wall (earth fill)
Crest height	10.0 ft
Length	201 ft
Maximum height	10.5 ft
Freeboard above maximum stage	3.5 ft
Control Structure	
Type	Concrete
Sill height	+1.0 ft
Net length of spillway crest	36.0 ft
Height of piers	10.0 ft
Sluiceways	
Width	5 ft
Number of bays	5
Height of stop logs at normal pool	6.2 ft
Log sluice width	11.0 ft
Fishway width	5 ft
Discharge channel capacity	950 ft/sec (est.)
Spillway Apron	
Type	Concrete
Length	73 ft
Width between abutments	68 ft 11 in.
Flood height	+1.0 ft
Bridge Over Control Structure	
Type	Public roadway
Height of roadway	11.22 ft
Roadway width	12.0 ft
Walkway height (for placing stop logs)	10.0 ft

Table 3

Pressuremeter Probe Test Data

Hole	Test	Probe Location Below Bottom of Monolith (ft)
G-P2	Test 1	3.9
	Test 2	7.9
	Test 3	11.9
G-P4	Test 1	4.2
	Test 2	8.2
	Test 3	12.2

Table 4

Split Spoon Data

	Depth Into Foundation (ft)		Blows Per
Hole	From	To	6-in. Penetration
G-P2	0.3	0.8	3
	0.8	1.3	5
	1.3	1.8	9
	15.6	16.1	6
	16.1	16.6	5
	16.6	17.1	6
G-P4	0.9	1.4	6
	1.4	1.9	6
	1.9	2.4	8
	15.9	16.4	2
	16.4	16.9	2
	16.9	17.4	6

Table 5

Unconfined Compressive Concrete Strengths

Core Hole	Specimen	Unconfined Compressive Strength (psi)
G-P2	G-P2T	6200
	G-P2M	5800
	G-P2B	3300
G-P4	G-P4T	7500
	G-P4M	4400
	G-P4B	6000
	Average ≃ 5500	

Table 6

Patching Material for Cracking, Spalled Joints, and Holes

Material	Parts by Weight
Cement	100
Water	≈18 (adjust as needed)
Acrylic-Polymer	27
Fine Sand (Passing No. 30 Sieve)	150

Table 7

Surfacing Material for Concrete Surface Rehabilitation by Using a Thin Overlay

Material	Parts by Weight
Cement	100
Water	≈20 (adjust as needed)
Acrylic-Polymer	30

Table 8

Forces at Top of Pile by Conventional Analysis

Case Loading	Horizontal Load F_H (kips)	Number of Piles	Horizontal Load per Pile (kips)	F_V (kips)	e (from Moment Center) (ft)	Moment about Center of Gravity of Pile Group $F_V(9 - e)$ (kip-ft)	Moment of Inertia of Pile Group (ft^4)	Maximum Compressive Force per Pile (kips) P/N + Mc/I	Maximum Tensile Force per Pile (kips) P/N - Mc/I
Normal Operation	23.76	8	2.97	195.14	9.04	195.14(9 - 9.04) = -7.8	$4(2)^2 + (4)(6)^2 = 160$	24.4 + 0.4 = 24.8	None
Normal Operation With Truck Loading (H15-44)	23.76	8	2.97	219.14	8.14	219.14(9 - 8.14) = 188.5	160	27.4 + 10.6 = 38.0	None
Normal Operation With Earthquake	29.58	8	3.70	195.14	8.85	195.14(9 - 8.85) = +29.3	160	24.4 + 1.6 = 26.0	None
Normal Operation With Ice	68.76	8	8.60	195.14	7.31	195.14(9 - 7.31) = +329.8	160	24.4 + 18.6 = 43.0	None
High Water Condition	29.39	8	3.67	179.72	8.90	179.72(9 - 8.90) = +18.0	160	22.5 + 1.0 = 23.5	None

Table 9

Results of Three-Dimensional Pile Foundation Analysis at Gull Lake Dam

Load Case	Pile	Distance From Upstream Edge of Pier ft	Distance From Left Edge of Pier ft	Axial Load kips	Lateral Load kips	Vertical Deflection at Top of Pile in.	Horizontal Deflection at Top of Pile in.
Normal operation	1	16	1	24.10	2.97	0.0295	0.0078
	2	12	1	24.30	2.97	0.0298	0.0078
	3	8	1	24.50	2.97	0.0300	0.0078
	4	4	1	24.68	2.97	0.0302	0.0078
	5	16	5	24.10	2.97	0.0295	0.0078
	6	12	5	24.30	2.97	0.0298	0.0078
	7	8	5	24.50	2.97	0.0300	0.0078
	8	4	5	24.68	2.97	0.0302	0.0078
Normal operation with truck loading (H15-44)	1	16	1	34.46	2.97	0.0422	0.0078
	2	12	1	29.75	2.97	0.0365	0.0078
	3	8	1	25.04	2.97	0.0307	0.0078
	4	4	1	20.32	2.97	0.0249	0.0078
	5	16	5	34.46	2.97	0.0422	0.0078
	6	12	5	29.75	2.97	0.0365	0.0078
	7	8	5	25.04	2.97	0.0307	0.0078
	8	4	5	20.32	2.97	0.0249	0.0078
Normal operation with earthquake	1	16	1	25.49	3.70	0.0312	0.0098
	2	12	1	24.76	3.70	0.0303	0.0098
	3	8	1	24.03	3.70	0.0294	0.0098
	4	4	1	23.29	3.70	0.0285	0.0098
	5	16	5	25.49	3.70	0.0303	0.0098
	6	12	5	24.76	3.70	0.0294	0.0098
	7	8	5	24.03	3.70	0.0294	0.0098
	8	4	5	23.29	3.70	0.0285	0.0098
Normal operation with ice	1	16	1	36.76	8.60	0.0450	0.0227
	2	12	1	28.52	8.60	0.0349	0.0227
	3	8	1	20.27	8.60	0.0248	0.0227
	4	4	1	12.02	8.60	0.0147	0.0227
	5	16	5	36.76	8.60	0.0450	0.0227
	6	12	5	28.52	8.60	0.0349	0.0227
	7	8	5	20.27	8.60	0.0248	0.0227
	8	4	5	12.02	8.60	0.0147	0.0227
High water condition	1	16	1	23.14	3.67	0.0284	0.0097
	2	12	1	22.69	3.67	0.0278	0.0097
	3	8	1	22.24	3.67	0.0273	0.0097
	4	4	1	21.79	3.67	0.0267	0.0097
	5	16	5	23.14	3.67	0.0284	0.0097
	6	12	5	22.69	3.67	0.0278	0.0097
	7	8	5	22.24	3.67	0.0273	0.0097
	8	4	5	21.79	3.67	0.0267	0.0097
Allowable	--	--	--	124	8.50	0.2500	0.2500

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Pace, Carl E.
Structural stability evaluation Gull Lake Dam : final report / by Carl E. Pace (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, [1981].
23, [30] p. : ill. ; 27 cm. -- (Miscellaneous paper / U.S. Army Engineer Waterways Experiment Station ; SL-81-16)
Cover title.
"June 1981."
"Prepared for U.S. Army Engineer District, St. Paul under Intra-Army Order No. NCS-1A-78-75."
Bibliography: p. 23.

1. Concrete piling. 2. Dams--Foundations. 3. Gull Lake Dam (Minn.) 4. Structural stability. I. United States. Army. Corps of Engineers. St. Paul District. II. U.S. Army Engineer Waterways Experiment Station. Structures Laboratory. III. Title IV. Series:

Pace, Carl E.
Structural stability evaluation Gull Lake Dam : ... 1981.
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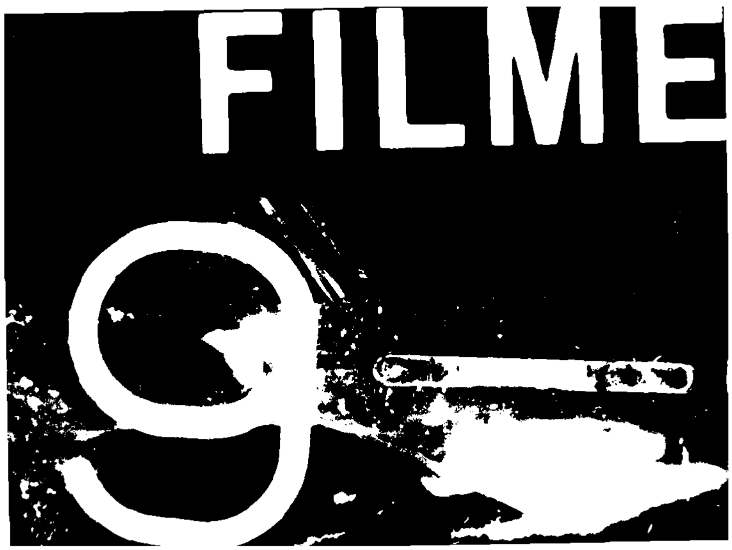
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